# Engineering News

A Quarterly Journal of the Pakistan Engineering Congress

Vol: 41 March 2003 No: 3

# PAKISTAN ENGINEERING CONGRESS

## THE EXECUTIVE COUNCIL FOR THE 69$^{TH}$ SESSION

**PRESIDENT**

Engr. Ch. Muhammad Rashid Khan

**VICE PRESIDENTS**

| | | | |
|---|---|---|---|
| 1 | Engr. Riaz Ahmad Khan | 8 | Engr. Mian Shamim Haider |
| 2 | Engr. Malik Ahmed Khan | 9 | Engr. Syed Akhtar Ali Shah |
| 3 | Engr. Muhammad Arif | 10 | Engr. Syed Anwar ul Hasan |
| 4 | Engr. Masood Ali Khan | 11 | Engr. Syed Mansoob Ali Zadi |
| 5 | Engr. M. A. H. Enver | 12 | Engr. Faqir Ahmed Paracha |
| 6 | Engr. Dr. Izhar ul Haq | 13 | Engr. Abdur Raziq Khan |
| 7 | Engr. Prof. Syed Ali Rizwan | 14 | Engr. Ikram ul Haq |

**OFFICE BEARERS**

| | | | |
|---|---|---|---|
| 1 | Engr. Ch. Ghulam Hussain | . . . | *Secretary* |
| 2 | Engr. Muhammad Siddiq Chattha | . . . | *Joint Secretary* |
| 3 | Engr. M. Ijaz Ahmad Cheema | . . . | *Treasurer* |
| 4 | Engr. M. Anwar Qaseem Qureshi | . . . | *Publicity Secretary* |
| 5 | Engr. Husnain Ahmad | . . . | *Business Manager* |
| 6 | Engr. Rana Muhammad Saeed Ahmad Khan | . . . | *Chief Editor Engg. News* |

**EXECUTIVE COUNCIL MEMBERS**

| | | | |
|---|---|---|---|
| 1 | Engr. S. N. H. Mashhadi (Late) | 19 | Engr. Nayyar Saeed |
| | (Immediate Past President) | 20 | Engr. Pir M. Jamil Shah |
| 2 | Engr. Ch. Aftab Ahmad Khan | 21 | Engr. R. K. Anver |
| 3 | Engr. Ahmad Nadeem | 22 | Engr. Sabir Khan Sadozai |
| 4 | Engr. Anwar Ahmad | 23 | Engr. Syed Nafasat Raza |
| 5 | Engr. Capt. (Retd) M. Qadir Khan | 24 | Engr. Muhammad Tahir |
| 6 | Engr. Ch. Khuda Yar | 25 | Engr. Raheel Sultan |
| 7 | Engr. Muhammad Akhtar Chaudhary | 26 | Engr. Sajjad Hussain Nasim |
| 8 | Engr. Ch. Muhammad Shafiq | 27 | Engr. Tahir Sultan |
| 9 | Engr. Habib ul Haq Randhawa | 28 | Engr. Imtiaz Ahmad Paracha |
| 10 | Engr. Iftikhar Ahmad | 29 | Engr. Ikram ul Haq Siddiqui |
| 11 | Engr. Imtiaz Aziz Bhatti | 30 | Engr. M. P. Gangwani |
| 12 | Engr. Khalid Javed | 31 | Engr. Syed Abdul Qadir Shah |
| 13 | Engr. Khawaja Akhtar Abbas | 32 | Engr. Muhammad Saeed |
| 14 | Engr. Muhammad Anis Butt | 33 | Engr. Khalid Latif Khawaja |
| 15 | Engr. Muhammad Ibrahim Malik | 34 | Engr. Ch. Mazhar Ali |
| 16 | Engr. Muhammad Khalid Cheema | 35 | Engr. Qaiser Zaman |
| 17 | Engr. Najam Waheed | 36 | Engr. Mian Saeed ud Din |
| 18 | Engr. Naveed Alam | | |

## COVER PHOTO

The upper photo is the silt free water in Indus River downstream of Tarbela Dam.

The lower photo is that of an open drain in the Township locality of Lahore. Raw sewer is also pumped into this drain which joins up with Hadiara Drain and then on to Ravi River.

Photo and statement on the cover with the courtesy of **Engr. A. W. Mir.**

44th YEAR OF PUBLICATION

# Engineering News

A Quarterly Journal of the Pakistan Engineering Congress
Vol. 41 Nov 2002 - March 2003 No. 3



**ON OTHER PAGES**

**IN THIS ISSUE**

**EDITORIAL**



**NEWS**



**TECHNICAL VIEWS**



PUBLISHED BY
Pakistan Engineering Congress

**FOR MEMBERS ONLY**

**Pakistan Engineering Congress is an age old professional body dedicated, interalia to technical advancement of Science & Engineering in the country. This magazine is distributed free of cost to all concerned commensurate with PEC objectives.**

- ALL COMMUNICATIONS SHOULD BE ADDRESSED TO THE CHIEF EDITOR, ENGINEERING NEWS PEC BUILDING, LIBERTY MARKET, GULBERG-III, LAHORE.

- FREE TO MEMBERS OF PAKISTAN ENGINEERING CONGRESS.
  ANY CHANGE OF ADDRESS SHOULD BE PROMPTLY INTIMATED GIVING OLD AS WELL AS NEW ADDRESS ALONGWITH MEMBERSHIP NUMBER.

- CONTRIBUTION TO THIS JOURNAL IN THE FORM OF ARTICLES, NEWS ABOUT ENGINEERING WORKS, NEWS ABOUT ENGINEERS, PHOTOGRAPHS AND TECHNICAL DATA ETC. ARE CORDIALLY INVITED.

EDITORIAL

# ENVIRONMENTAL POLLUTION AND WATER CONSERVATION

About $^{2}/_{3}$ of the earth's surface in covered with water. However, nearly 97% of planet's water is salt water in seas and oceans, while close to 2% water is frozen in polar ice sheets and glaciers and hardly less than 1% of fresh water is available which is fit for human consumption. Six billion people on the earth's surface have already overtaxed this available and accessible supply of fresh water, resultantly the situation is aggravating rapidly particularly in the developing countries and we are a part of this group.

Some of the salient environmental issues confronting mankind in the 21st century include green-house effect, global warming, ozone depletion, deforestation, acid rain. Indiscriminate discharge of raw sewage, untreated industrial wastes and solid wastes into the water courses has given rise to gross degree of water pollution, resulting in destruction of aquatic life including both flora and fauna. It renders the water quality unfit for human uses and adversely affects the fishing industry. It is also polluting ground water at many places. Obnoxious emissions in the air from industries, vehicles and thermal power plants are contaminating air, resulting in serious health hazards besides large scale economic losses particularly in the developing world. Noise pollution is also taking its toll particularly in the urban areas.

It is quite unfortunate that whatever meagre resources of fresh water we have in our country, we are ruining them at a fast pace. For example raw sewage and untreated industrial wastes are being dumped into the river Ravi which is destroying marine life and polluting under-ground water of hand pumps and shallow wells located in villages in vicinity of the river bank. Ground water is being extracted for various uses in an uncontrolled manner, resulting in lowering of water-table and intrusion of brackish water into sweet water zones. No measures are taken to artificially recharge ground water aquifer due to lack of sources for natural recharging. No serious and constructive effort is being made to conserve and protect surface water by storage in large reservoirs. No big dam has been constructed in the country for the past few decades, while our two major dams already constructed viz. Mangla and Tarbela are rapidly filling up with silt and reducing its useful storage capacity.

Urgent and meaningful actions at national level are needed to control pollution and for water conservation. Stringent measures need to be taken to control ever increasing pollution problems by enforcement of environmental pollution laws of the country. Furthermore, educating masses through electronic media to create awareness at grass root level is urgently needed.

*********************

## WELCOME TO NEW MEMBERS

*The Executive Council of the Pakistan Engineering Congress approved Membership of the following new members in to the Congress fold. The Engineering News congratulates all of them and welcomes to PEC*

**Members admitted on 17-8-2002**

1 Engr. Khalid Rashid Qureshi
2 Engr. Naeem-ur-Rehman Akhoond
3 Engr. Dr. Tahseen Rahim
4 Engr. Ghulam Abbas Cyclewala
5 Engr. Rear Admiral (R) M. I. Arshad
6 Engr. Nadeem Akhtar Sheikh
7 Engr. Kamruddin Vali
8 Engr. Khalid Pervez
9 Engr. M. Amir Saeed
10 Engr. Azeemullah
11 Engr. Muhammad Tariq
12 Engr. Hidayatullah
13 Engr. Muhammad Raqeeb
14 Engr. Madni Ahmed Ali Arfat
15 Engr. Akbar Ali
16 Engr. Mehddi Hasan
17 Engr. Shazia Baqar
18 Engr. Farhan Badar
19 Engr. Muhammad Hayat Bhangar
20 Engr. Muhammad Saleem
21 Engr. Mansoor Ahmed
22 Engr. Ali Ahmad
23 Engr. Saleem Ahmad Sheikh
24 Engr. Tahseen Ahmad Sheikh
25 Engr. S. M. Larik

**Members admitted on 21-9-2002**

1 Engr. Mahmood Shah Muhammad
2 Engr. Andul Rashid Awan
3 Engr. Muhammad Tanveer Ishaq
4 Engr. Dr. Shahid Ahmad

**Members admitted on 3-11-2002**

1 Engr. Muhammad Tahir
2 Engr. Muhammad Javed Akhtar Butt
3 Engr. Hafiz Liaqat Bashir
4 Engr. Fasial Mahmood
5 Engr. Kamran Kashif Rana
6 Engr. Muhammad Imran Majeed
7 Engr. Sh. Maqbool Hasan Ali
8 Engr. Niamat Khan
9 Engr. Miss Humaira Javed

**Members admitted on 28-12-2002**

1 Engr. Nadeem-ur-Rehman Baig

TECHNICAL VIEWS

# THE VOID POROSITY OF ROCK ARMOUR IN COASTAL STRUCTURE

By

**J. – P. Latham, S. Newberry, M. Mannion, J. Simm and T. Stewart**

Measurement and payment issues are particularly contentious when dealing with rock armour structures. The paper elaborates upon the fundamental controls of rock fragment packing and progresses to consider rock armour packing and the financial consequences of errors in void porosity assumptions. Results from detailed case studies, construction of full-scale experimental revetments and numerous contracts are summmarised in order to compile a data source from which up-to-date guidance on void porosity and layer thickness coefficients can be drawn. A new rapid survey method to assess in-situ packing density and to compare armourstone packing between surfaced panels is introduced.

## NOTATION

| | |
|---|---|
| $A$ | area of trial panel in plane of armour layer [m$^2$] |
| $C$ | Gupta's shape coefficient |
| $D_{n50}$ | equivalent cubic dimension of 50% passing block in an armour grading |
| $K_t$ | layer thickness coefficient |
| $L$ | shore-parallel chainage length in panel [m] |
| $m$ | uniformity exponent in power law size distribution |
| $n$ | number of layers considered in a rock armour panel |
| $n_v$ | void porosity |
| $n_{vtp}$ | void porosity determined from trial panel survey |
| $n_{vd}$ | target void porosity assumed in a design |
| $N$ | number of blocks on a panel of armourstone |
| $Q$ | number of blocks placed in armour layer per unit area in plane of armour layer [m$^{-1}$] |
| $Q'$ | dimensionless packing density of armour blocks (in a trial panel or in surface layer of a panel) |
| $R$ | Power's Roundness |
| $T_d$ | orthogonal armour layer thickness [m] |
| $V_d$ | design bulk volume of armour layer [m$^1$] |
| $V_r$ | solid rock volume in armour [m$^1$] |
| $V_{r1}$ | solid rock volume in armour layer calculated from design parameters [m$^1$] |
| $V_{r2}$ | solid rock volume in armour layer calculated from surveyed parameters [m$^1$] |
| $V_s$ | surveyed bulk volume of armour [m$^1$] |
| $W_m$ | mean mass of blocks in an armour grading [1] |
| $W_{50}$ | 50% passing block mass in an armour grading [1] |
| $x$ | sieve size [mm] |
| $x_{100}$ | 100% passing sieve size [mm] |
| $x_{av}$ | average sieve size in Gupta's sized gradings [mm] |
| $y$ | fraction by mass lighter than a given sieve size |
| | cross-sectional area normal to chainage line (m$^2$) |
| $\rho_r$ | rock density [tm$^{-3}$] |

## 1. INTRODUCTION

Rock armour is one of the most common construction materials to be in the front line of wave action on coast protection structures owing largely to its massiveness, durability, flexibility and low cost. However, its innate irregularity in geometry poses engineering problems by introducing variability and uncertainty into the final structure. This paper examines void porosity as this important property is highly dependent on the natural irregularity of rock armour sizes and shapes.

There are two main reasons why packing density and the associated void porosity are important to coastal engineers. The first is that it affects hydraulic performance, because of energy dissipation occurring in the voids, which in turn affects wave reflections, stability, run-up and overtopping. The second is its relation with materials procurement. At the design stage, an understanding of packing relationships is needed for estimating armour layer thickness and for dimensioning the cross-section design drawings to be used by the contractor. Furthermore, the total tonnages of rock that must be ordered to make up the bulk volumes indicated on the drawings depend on the void space and thus the packing density. Uncertainty about voids and packing in the built structure leads to greater risk for both designer and contractor. The most direct financial risks are those associated with procurement and payment issues while potentially more important but indirect financial risks relate to uncertainty in hydraulic performance.

The paper does not consider the hydraulic implications of the variations in packing density and voids but will discuss the void space within armour layers and their layer thicknesses. Topics covered include factors affecting rock fragment packing in general, rock armour packing, rock payment issues, previously published and recentresearch data from several full-scale structures and finally, a new but simple method for on-site evaluation of rock armour packing.

## 2. ROCK FRAGMENT PACKING

In general, the factors affecting the void porosity of any rock fragment pack can be summarised as particle size distribution (PSD), distribution of shapes, container wall effects, absolute size and strength effects, surface roughness, deposition history, loading history (including pseudo-static, dynamic and hydraulic loading), and in time, durability.

Peronius and Sweeting presented a prediction model for 'minimum porosity' with shape effects quantified by visual comparison using Power's Roundness parameter [see Fig. 1] and with PSD effects accounted for by a parameter given by the maximum vertical (i.e. % passing) deviation from the equivalent Fuller parabola. The equivalent Fuller parabola is a theoretical grading curve according to equation (1), with m = 0.5, passing through the 50% passing size of the PSD in question. An illustration of their minimum porosity prediction model has been explored in Fig. 2 for a range of exponents m in the power law size distribution.

$$y = (x/x_{100})^m \qquad (1)$$

Equation (1) defines the family of Fuller curves where y is the fraction by weight finer than a given sieve size, x is the sieve size and $x_{100}$ represents the 100% passing (i. e. the maximum) sieve size. An increase in the exponent, m, indicates greater uniformity in sizes.

Power's Roundness values may range from R ~ 0.14 (very angular) to R ~ 0.84 (well rounded). The prediction of Peronius and Sweeting, explored for power law PSD curves, is shown in Fig. 2. It is calibrated with sand to gravel sized particles from an experimental method for filling test cylinders that results in tight packs and is thus termed minimum porosity. Unfortunately, agreement between different porosity prediction models appears poor and accuracy for predictive calculation based on any such model is likely to remain poor when applied to the varieties of PSD, particle shapes and amounts of compaction found in practice.

In a study of hydraulic friction in granular materials of different shape and size the percentage porosity $n_r$ was investigated as a function of average size for single-sized aggregates from 6 mm to 100 mm, making sure the wall effects of the cylindrical container were negligible. Equation (2) was obtained for these single-sized gravels.

$$n_v = cx_{av}^{-0.032} \qquad (2)$$

Where $n_v$ is the void porosity, $x_{av}$ is the average particle size in mm, C varies with the average shape of the particles which were represented by values of 45.43 for angular limestones, 41.44 for sub-angular crushed quartzite, 37.89 for rounded gravel and various values for proportional mixtures of rounded gravel and crushed aggregates. This relation implies an absolute size effect for the gravity-dominated packing of relatively single-sized particle assemblies if it is reasonable to assume the shape characteristics were constant for all the different sized samples classed together e.g. as angular limestone. In extrapolating equation (2) substantially beyond its applicable range to armour-sized particle assemblies (1 metre blocks), substituting the shape coefficient for angular limestone gives a void porosity of 36.42% (30.38% for rounded gravel shapes).

In many branches of engineering, problems in correctly characterizing the degree of compaction and the shape characteristics applicable are often encountered so that in-situ tests are invariably preferred to guidance from tables and prediction models. In the next 10 years however, it is considered a realistic hope that discrete element models will be able to model the packing of real shaped particles.

## 3. ROCK ARMOUR PACKING

Consider now the factors influencing packing of rock of armour as itemised in the list below and illustrated in Fig. 3.

- (a) Grading
- (b) Shape
- (c) Placement method (machine, instruction, time)
- (d) Definition of bounding volume
  - (i) Surface survey method
  - (ii) Underlayer survey
  - (iii) End-point profile error
- (e) Settlement
- (f) Other (e.g. strength, compaction, hole filling)

Certain differences from the factors identified in the previous section concern human influences. As with rock fragments higher voids occur with a more single sized grading. The shape, if very blocky, will often lead to tighter packing. Placement method is constrained by the handling machine's grab type, the kind of rock armour surface finish requested and the experience of the machine operators and time constraints. How to define the bounding volume is perhaps the largest source of confusion and this can make a big difference. Settlement occurring between surveying the underlayer and the armour surface has also often been suspected but rarely quantified. To the author's knowledge, it has yet to be found a significant cause of underestimating rock armour volumes. Other factors include strength. A weaker rock will have corners more easily broken off, thus encouraging lower positioning and tighter packing. Armour may also be driven over and slightly compacted during construction. In certain more accessible areas used for recreation, the bigger voids present a safety hazard and are often filled to try and reduce accidents. The problems of void porosity determination are further highlighted in Fig. 3 and the factors are discussed below.

In the schematic cross-section of a revetment (e.g. for rehabilitation of a sea-wall with rock armoured revertment and beach recharge) shown in Fig. 3, the numbers refer to the factors in the list given above. The cross-sectional area presented on a design drawing (straight boundary) differs from that surveyed by conventional levelling staff (irregular boundary). The figure illustrates how blocky shapes tend to sit flat down on the underlayer and form closer packs if placement method is not deliberately inhabiting this (shown in the circles numbered 3). Different surface surveys are shown coinciding in 4a, while 4b shows the underlayer surface falling below the design drawing levels. In 4c the position of the end point of the surveyed profile is shown to be quite critical, especially with short profiles and automatic volume calculation using 3D surface computer packages (e.g. see Fig. 7g), when such sources of error can be obscured. A last survey point should therefore be judiciously added to those measured on a regular spacing interval. Fig. 3 also draws attention to differences between design drawings and actual construction. Some times a double layer will be specified in the construction method and shown explicitly in the design drawings, but the only practical way to achieve these specified layer thicknesses as shown on the drawing, is to build a layer three blocks thick. This will often arise as a result of poor choice of $k_t$ values when applying the layer thickness calculation (see section 4).

## 4. PAYMENT ISSUES

Figure 4 shows a prismatic tabular section of a rock armour layer. There are two bulk volumes to consider. Before construction the design bulk volume, $V_d$, is required. This volume equals the area A times the orthogonal thickness $T_d$, which is a theoretically predicted average thickness that the engineer has introduced for the design drawings, using recommended layer thickness coefficients. Clearly, for a given range of armourstone weights or sizes, the best design guidance on the expected single or double layer thickness is required. These volume calculations are then helpful for ordering the necessary tonnages prior to construction. After construction, in addition to profile tolerance conformance checks, it may be necessary to determine volumes for payment purposes. Having surveyed the underlayer surface and the armour layer surface, the average cross-sectional area. $_s$, times the chainage length L gives the surveyed bulk volume, $V_s$. From and the down slope length, an actual average orthogonal thickness can be deduced and this often makes an interesting comparison with the orthogonal thickness shown on the design drawings. Large differences are often the cause of disputes, some times leading to contractual problems.

Of the two main types of contracts in the UK, the payment-by-weight contract seems most reasonable to the contractor since he/she pays for the rock by weight and if he can prove that this weight has been used without unreasonable wastage, he/she is remunerated pro-rata for that weight used. However, the client does take more risk. The rock may end up with a tight pack that reduces hydraulic efficiency and promotes high run-up. Examples have been reported of this occurring in Hong Kong (e.g. at the Annual One Day Meeting of the UK Armourstone User's Group held at HR Wallingford). Considerably more rock than is necessary may be crammed in, and all this rock has to be paid for by the client.

Payment-by-volume contracts are often used in the UK. This is because tonnages supplied by the sea are usually verified only by water-displacement surveys for barge unloaded and barge loaded and clients often do not find this sufficiently reliable, preferring to work backwards from surveyed bulk volumes to weights using an assumed void porosity. The contractor takes a risk with such contracts, because the conversions, can give tonnages short of the amount required and actually purchased. Let us examine such contracts a little further by considering how the solid volume of rock can be determined. Neglecting an allowance often given for breakage and losses (not to be mentioned further as it will confuse the main issues), the volume of solid rock $V_r$ the client is prepared to pay for is obtained from the bulk volume $V_b$ and a void porosity $n_v$. Both bulk volume and void porosity can be set in different ways.

The contract may indicate that the 'design target porosity' $n_{vd}$ is to be used for payment purposes where the specifier has taken a porosity value from a guidance table or from previous experience. Provision for the construction of an early trial panel in the contract is becoming more accepted in the UK ever since 1991 when the practice was suggested in the CIRIA/CUR rock manual, field measurements from a weighed and surveyed initial trial section of the structure are analysed. If survey results are acceptable to the client/engineer, the panel sets the precedent for agreed construction practices for the rest of the project and results can also provide an actual void porosity calculation $n_{vtp}$ upon which payment can be based. It is also suggested that the field techniques, introduced in section 9, together with their associated assumptions and provisos, may provide, a practical means by which the estimated void porosity within different areas of the structure can be compared with the target value and tolerance bandwidth for void porosity – normally laid down in the contract.

If the rock volume is based on design drawing bulk volumes $V_d$ and design target porosities, $n_{vd}$, provided the final profiles are met within tolerances, the client is usually happy with the resultant calculated rock volume $V_{r1}$. If the bulk volumes are surveyed bulk volumes, $V_s$, and the void porosity is obtained from trial panel results using the same survey techniques then the contractor will usually be happy as this calculated rock volume, $V_{r2}$, must give one of the best determinations of the actual rock volume that has been placed. These differences are summarized as follows :

$V_{r1} = V_d\,(1-n_{vd})$ least client risk

$V_{r2} = V_s\,(1-n_{vtp})$ least contractor risk

Indicating that bulk volumes and void porosities can be specified differently and there are many combinations and variations on the two extreme approaches.

Clearly, before construction, engineers and contractors need to know the expected value and likely variation of the value for $V_r$, which depends on the layer thickness coefficients used to calculate $V_d$ as well as $n_{vd}$. Survey methods and layer thickness coefficients must therefore be considered carefully.

For the double layer of rock armour placed on an underlayer as shown in Fig. 5, the squares (or cubes in 3D) represent the equivalent cubic size of each block. Thus a line is drawn to represent an idealized $2D_{n50}$ thick layer. In practice, the layer is surveyed and often found to be somewhat lower. How much lower depends on the survey definition of the surface. Three different lines are shown. The layer thickness coefficient $k_t$ determines how much lower (or higher) but the value chosen for $k_t$ is somewhat meaningless if the survey method is not indicated. The figure shows how for surveys with highest points on all blocks under a traverse line, $k_t$ will be larger, giving greater thicknesses To survey the surface profiles, the CIRIA/CUR rock manual introduced a half $Dn_{50}$ sized hemispherical probe and a standard survey spacing to clear up ambiguities in applying recommended $k_t$ and $n_v$ values. A conventional staff will penetrate even further into the voids in the armour surface and give even lower $k_t$ values. The experience of the authors suggests that surveying at regular intervals is best achieved by first constructing a layer-parallel frame line in the profile plane. The survey intervals are marked off and then the staff is guided vertically down onto the rock surface as objectively as conditions allow. Clearly, there is scope to adopt a high point survey method as the standard reference method. However, such a method is not compatible with a survey technique that can be applied under water, whereas the survey method using the half $Dn_{50}$ sized hemispherical probe and a standard horizontal spacing interval can be used indentically above and below water (a circular cage on the end of a crane line is often used for 'soundings'). The half $Dn_{50}$ sized foot method is also the basis of most automated profiling systems in hydraulics laboratories. It should be noted that surveying in difficult weather conditions and on slippery rocks with steep slopes raises questions regarding personnel safety that must be addressed responsibly.

In the Reculver study of a revetment built with a double layer of 1 to 3t armour (see section 6), 13 survey profiles were obtained using each of the three survey methods discussed. The sample interval, was 0.5 m for each profile except for the high point method where heights are determined once only per block traversed. Average layer thickness for each profile was determined and a mean thickness of the panel was derived from the 13 average profile thicknesses. Mean and standard deviation from the 13 profiles were expressed as follows : $1.547 \pm 0.167$ m, $1.717 \pm 0.108$ m and $1.829 \pm 0.092$ m for the conventional staff, the half $Dn_{50}$ foot and high point methods respectively (coefficients of variation of 10.8%, 6.30% and 5.0% respectively). While these results do not assess method variability from repetition along the same profile surface, they do suggest that the high point method may give marginally better repeatability than the $Dn_{50}$ foot method for assessing the average thickness of a panel. However, the standard deviation of thicknesses recorded along each individual profile tends to be somewhat higher for the high point method and for reasons given above, the high point method is not the preferred survey method.

## 5. PREVIOUS WORK – BEESANDS 1992

In the detailed study of packing in the Beesands revetment built with 4 to 9 t armourstone, the payment implications of using different contracts were investigated together with the effect of using different survey methods to define the bulk volume. The findings are summarized below.

(a) Payments based on weight :
notional cost of placing 1533 t of test panel armour at £ 25/t :
£ 38K for panel, 1.17M extrapolated for whole revetment.

(b) Payments based on volume.

| Profile measurement method | Mean layer thickness $T_s$ (m) | Computed £ Payments for rock volume $V_r$ $n_{vd}$ : 35% | $n_{vd}$ : 40% |
|---|---|---|---|
| High spot leveling staff | 2.38 | 39K 1.20M | 36K 1.11M |
| $0.5D_{n50}$ foot | 2.19 | 36K 1.10M | 33K 1.02M |
| Conventional levelling staff | 2.04 | 34K 1.03M | 31K 0.95M |
| Design thickness Td | 2.40 | 40K 1.21M | 37K 1.12M |

For a payment-by-weight contract, the contractor pays and receives £ 1.17M. For a payment-by-volume contract using the preferred $0.5D_{n50}$ foot profile measurement method, the contractor pays £ 1.17M but receives in accordance with the target porosity e.g. £ 1.10M if the target porosity was 35% -- or £ 1.02M for a target porosity of 40%. The actual porosity was determined to be 31.3% for the test panel. Certainly, these differences of £70.000 or £150.000 could cut out a substantial part of the contractor's profit margin and can run into millions outstanding for a very large contract. This is discussed more fully in an earlier paper, where the influence of survey method on payment is illustrated.

## 6. FIELD STUDY (1999) AT RECULVER, KENT

Figure 6(a) shows Reculver Castle in the background, which sits behind a Roman Wall protected by a substantial 3-6 t rock revetment. The structure seen in the foreground is a sea wall with rock armour toe protection. The small extent of the cross-section (b) and the 1 to 3 t armour grading made this an ideal full-scale structure to perform a detailed assessment of layer thickness coefficients and void porosity. This study of a small cross-sectional area revetment reported by Newberry enables a comparison of results with those of the Beesands structure (f) to be presented. The research used same three survey methods including the $0.5D_{n50}$ hemispherical foot at the base of the survey staff (c) as in the Beesands study. Void porosity and thickness parameters for the same three survey methods were evaluated from 13 profile sections [e.g. (e)] and the 240 blocks placed in the survey test panel were individually weighed (d). The trends for the three different survey methods are as expected in each structure, but the actual values are different for the two structures. A large orange peel grab was used in both cases, but the Beesands structure with heavier armour has yielded lower $k_t$ values, perhaps because the armourstone blocks are more difficult to prevent from lying with their shortest dimension upwards. Also, the larger blocks have given a tighter pack (see Table1).

## 7. FULL-SCALE TEST REVETMENT STUDY

Phase 1 of a collaborative study (DETR/HR/IC) requiring the assistance of contractors and suppliers, considered only full scale structures with the aim of studying the influence of shape, size and placement on $k_t$ and $n_{tr}$. Work from Phase 2 and 3 is to consider dry model structures and model structures under wave action. In Phase1, four full-scale artificial structures were carefully constructed between July and October 1999, each on a one-on-two slope. The $0.5D_{n50}$ staff survey method was used throughout. Single and double layers were studied and attempts to make dense packs were compared with standard packs deemed typical of UK construction practices. Different characteristic shapes were involved, the blocky shape of large Larvik armourstone being evident in Fig. 7(c), (d) and (e) while Larvik blocks of about 0.5 t at Immingham (f) were remarkably less blocky.

Figure 7(a) shows granite blocks in Bardon Hill aggregate quarry while (b) shows Carboniferous limestone in Torr Works quarry, both with a 3-6 t single armour layer ; (c), (d) and (e) show the construction of an experimental revetment with 8-12 t armour on the Shoreham foreshore near to where rock groynes have been recently completed. Note the tendency for larger blocks to be laid with shortest dimension normal to the slope because of difficulties in handling (c) and the relative ease with which contractors can produce smooth tight outer slopes with the blocky Larvik armourstone. A bucket grab was used at Immingham to place the ~ 0.5 t Larvik armourstone in yet another experimental revetment, sometimes with more than one piece at a time (f). All structures were surveyed and volumes analysed using standard software as illustrated by the digital underlayer surface and second armour layer surface for the Bardon Hill structure (g). Results from these four double layer test revetments plus the double layer Beesands and Reculver structures are summarized in (Table 1).

**Table 1. Summary of full-scale packing parameters using 0.5 $D_{n50}$ survey staff**

| Location | Beesands | Reculver | Shoreham | Immingham | Bardon | Torr Works | Mean |
|---|---|---|---|---|---|---|---|
| Grading (tones) | 4-9 | 1-3 | 6-12 | ~ 0.5 | 3-6 | 3-6 | ---- |
| Void porosity $n_v$ | 31.3 | 34.4 | 30.3 | 40.1 | 34.4 | 34.8 | 34.2 |
| Layer thickness Coeff, $k_t$ | 0.85 | 0.94 | 0.77 | 0.92 | 0.98 | 0.91 | 0.88 |

From the six structures undergoing detailed analysis with a $0.5D_{n50}$ survey staff, the porosity $n_v$ averaged 34% and $k_t$ averaged 0.88. Note that Shoreham represented a lowest case for both parameters, while Immingham gave the maximum void porosity. When attempts were made to produce a dense pack at three sites, the void porosity decreased to 28% at Shoreham, 31% at Bardon Hill and remained constant at 33% at Torr works, suggesting that contractors are unlikely to be able to bring about a reduction in the volume of voids of greater than 10% when compared with the volume of voids for standard construction methods.

## 8. COMBINED QUESTIONNAIRE AND FULL-SCALE STUDY DATA

Research data were gathered using questionnaire methods and follow-up discussions with engineers and contractors. Many but not all structures provided data that was based on $0.5D_{n50}$ staff surveys. Some of the data are based on surveys of the whole structure volume and others on trial panels. The new DETR/HR/IC study data have been added to the original set of structures in order to provide statistics for the key parameters. However, the combined data (Table 2) are from a mixture of survey techniques and contract types. Surprisingly, significant difference in the average $n_v$ for contracts specified by weight rather than by volume were not apparent. The average value for void porosity is 33.95% with a standard deviation of 2.83% (including the Immingham panel data for very small armourstone). It is therefore considered reasonable to infer that only one in 20 structures is likely to fall outside the void porosity range 28% to 40%.

**Table 2. Examples of site parameters and summary statistics**

| Site | Rock density | As-placed packing density ( t / m³ ) | Void porosity $n_v$ ( % ) | Contract payment by W -- weight V – volume | If $n_v$ was based on a trial panel |
|---|---|---|---|---|---|
| East Coast 1 | 2.73 | 1.85 | 32.2 | V | N |
| East Coast 2 | 2.73 | 1.72 | 37.0 | V | N |
| South 1 | 2.65 | 1.71 | 35.5 | W | N |
| South 2 | 2.60 | 1.82 | 30.0 | W | N |
| South 3 | 2.65 | 1.72 | 35.1 | V | N |
| South 4 | 2.64 | 1.82 | 31.1 | V | N |
| South 5 | 2.69 | 1.78 | 33.8 | W | N |
| South 6 | 2.67 | 1.83 | 28.8 | V | Y |
| South 7 | 2.75 | 1.90 | 30.9 | W | N |
| South 8 | 2.65 | 1.65 | 37.7 | V | N |
| South 9 | 2.65 | 1.70 | 35.8 | V | N |
| South 10 | 3.10 | 2.00 | 35.5 | V | N |
| South 11 | 2.65 | 1.70 | 35.8 | V | N |
| Beesands | 2.76 | 1.90 | 31.3 | V | Y |
| N. Wales 1 | 2.66 | 1.81 | 32.0 | V | Y |
| N. West 1 | 2.72 | 1.77 | 35.0 | W | N |
| N. West 2 | 2.70 | 1.82 | 32.6 | V | Y |
| N. West 3 | 2.70 | 1.90 | 37.0 | W | Y |
| Immingham | | | 40.1 | | Y |
| Reculver | | | 44.4 | | Y |
| Shoreham | | | 30.0 | | Y |
| Barden | | | 34.4 | | Y |
| Torr Works | | | 34.8 | | Y |
| AVERAGE | | | 33.95 | | Y |
| ST. DEV. | | | 2.83 | | |

## 9. BLOCK COUNT PANEL SURVEYS FOR THE STUDY OF ARMOUR PACKING

The principle of the block count method is trivial. An area is marked out on a surface panel using stretched tapes and the area is calculated or is surveyed more accurately. A systematic method for counting all blocks deemed to be in the top layer of the area is adopted. The number per square meter or surface packing density should be reasonably constant if the material sizes and placement methods have not varied.

For rock armour in a double layer or single layer panel, the solid rock volume is given by the number of blocks in the panel, N times their mean weight, $W_m$ divided by the rock density, $\rho_r$. The total bulk volume is given by the panel area in the plane of the layer. A times the orthogonal layer thickness, $t_a$. Putting in the expressions for solid and bulk volume we get :

$$1 - n_v = \frac{N.\,(W_m/\rho_r)}{A.n.k_t.D_{n50}} \qquad (3)$$

which multiplying numerator and denominator of RHS by $D_{n50}$ cubed and assuming $W_{50}/\rho_r = D_{n50}^3$ gives.

$$1 - n_v = \frac{N.\, D_{n50}^2}{A} \cdot \frac{W_m}{W_{50}} \cdot \frac{1}{n.\, k_t} \qquad (4)$$

This can be written as.

$$1 - n_v = Q' \cdot \frac{W_m}{W_{50}} \cdot \frac{1}{n.\, k_t} \qquad (5)$$

In equation (5), Q′ is the dimensionless packing density which is equivalent to the packing density parameter $\phi$ used to measure the packing density of concrete armour units (e.g. accropode, coreloc). It is simply the block counts per unit area multiplied by the area $D_{n50}$ associated with the armour grading in question, thus making Q′ a dimensionless packing density based on the block counts per unit area Q′. Equation (5) indicates that if the layer thickness coefficient $k_t$ and the grading size and uniformity are constant from one area to another, then variations in Q′ will indicate variations in porosity.

Each data set shown in fig. 8 is from a different groyne or revetment on which a rock count survey and an assessment of the block weights in the panels was conducted using a conversion from measured block dimensions. Fig. 8 shows how Q′ tends to come down as $W_{50}$ increases but after normalizing the results, one groyne was shown to have significantly lower packing density Q′. It transpired that this groyne was the last one to be built and it was constructed under difficult tidal constraints. However, because of the small size of the panel, results may not have been representative of the whole groyne.

Block count surveys from 74 panels on 12 structures built from Larvik armourstone were made. After normalizing the block counts data, a comparison of packing density Q′ was obtained showing an average value of about 0.6 (Fig. 9). The exceptional values indicated for Easthourne and Sandgate can be discerned from such a plot and further study of these structures could reveal the most likely explanation, for example that the $W_{50}$ in the top layer of the panel locally differs from the specification value assumed for the plot.

Fortunately, there was very good control on the block counts and exact weights in the top layer of the panel for the Reculver structure and for the top and bottom layers of the standard and dense packs for most of the DETR/HR/IC study structures. The packing parameters including void porosity were further investigated using equation (5) as shown in Fig. 10. Q′ gives an approximately constant value averaging just below 0.6. Using the extra available data on $k_t$ values and mean weights, the void porosity in the armour layer of each panel can be predicted as shown in Fig. 10. For these full-scale study panels the predicted porosity using equation (5) can be compared with the actual porosity. The quality of the relationship is clear in Fig. 11. Predicted and actual porosity are within about 1.5%. The actual void porosity ranges from just below 40% to just below 30%. There is an apparent trend of falling porosity with increasing block size that appears significant. There are many possible reasons for this scale effect. They include the tendency for heavier blocks to be a different (e.g. blockier and more equant) shape and to undergo greater asperity crushing of corners during placement. It is interesting in this context to recall the empirical relation of equation (2) discussed earlier.

In practice, the $k_t$, $W_{50}$ and $W_m/W_{50}$ values in a panel are rarely measured, but as a first approximation, the contract values can be substituted to enable a check to be made on the void porosity. In summary, the block count survey method proposed is a simple and quick method of checking workmanship. There is also no reason why it could not be used with good photographs, for example as part of monitoring.

## 10. CONCLUSIONS

(a) $n_v$ for high point survey is less than $n_v$ for the $0.5D_{n50}$ staff survey by 4% to 6%.

(b) $k_t$ for high point survey is greater than $k_t$ for the $0.5D_{n50}$ staff survey by 0.04 to 0.07.

(c) Guidance for average void porosity and layer thickness coefficients based on the $0.5D_{n50}$ staff survey method using a limited but well controlled data set gives $n_v$ = 34% and $k_t$ = 0.88.

(d) Based on all reviewed full-scale structures the following statistical values are suggested for providing an indication of the mean value of void porosity and its variability based on over 20 different UK structures.

$n_v$ mean = 33.95% $n_v$ standard deviation = 2.83%

(e) Block count surveys can provide a simple method to check site workmanship and with care, using a newly developed and tested simple theory, can be used to draw conclusions about the packing density and void porosity.

*(Courtesy : Water and Maritime Engineering Vol. 154 issue 3)*

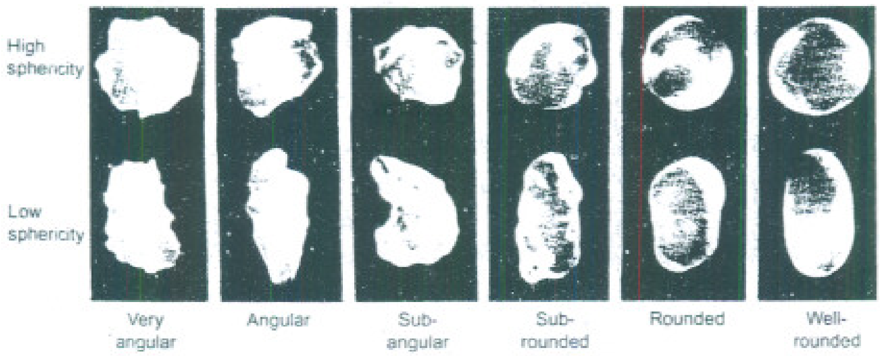



Fig. 1. Powers' Roundness illustrated with models photographed for visual comparison (adapted from the original paper)

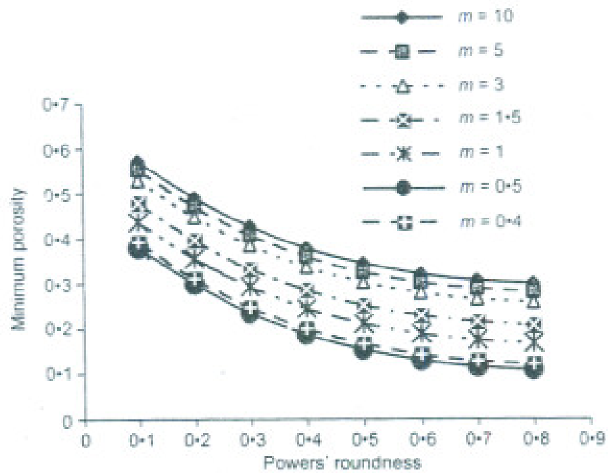



Fig. 2. Minimum porosity as a function of Powers' Roundness for various values of uniformity index $m$ in equation (1), based on the relation given in Peronius and Sweeting

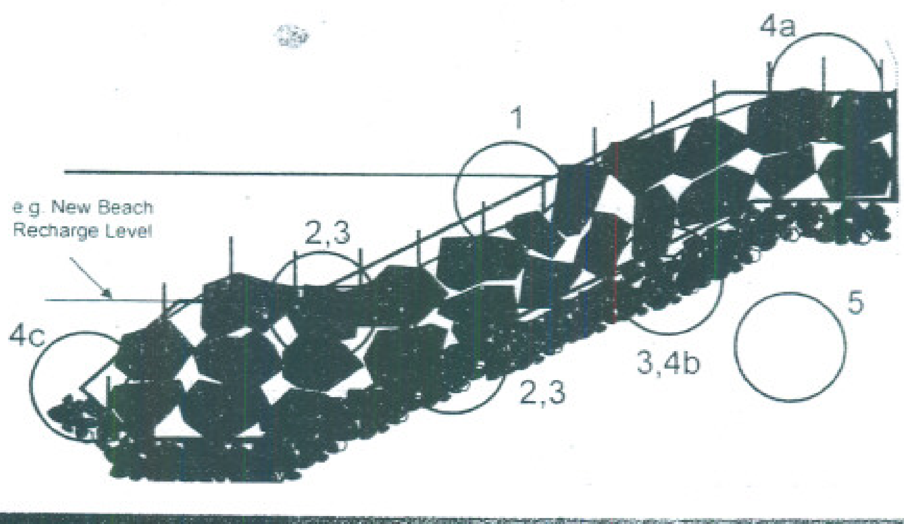



Fig. 3. Factors influencing rock armour thickness and void porosity measurements, the numbers refer to those listed in section 8 and discussed in section 9

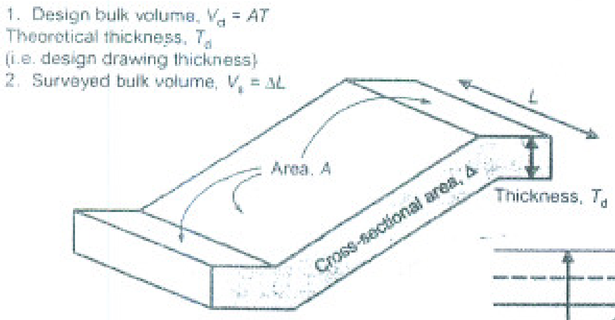



Fig. 4. Basis of volume calculations

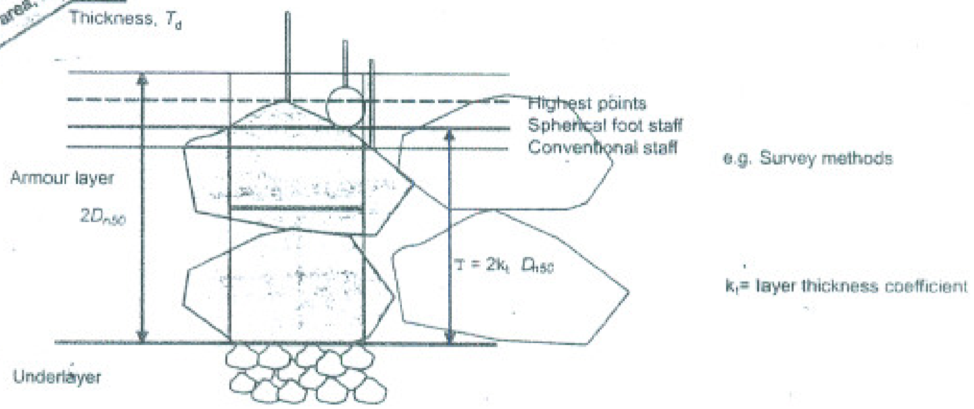
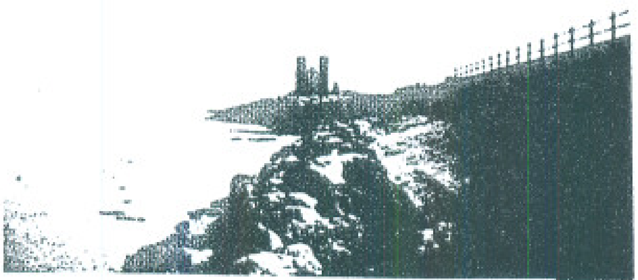
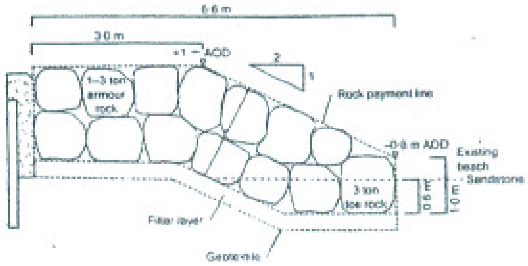



Fig. 5. Layer thickness as a function of survey method and layer thickness coefficient, $k_t$

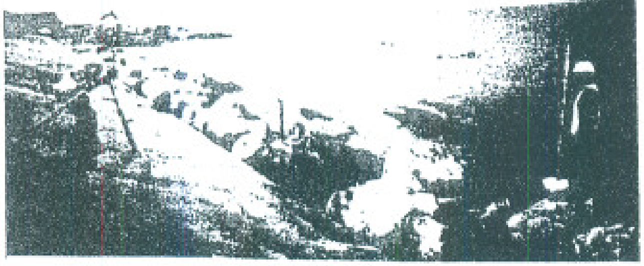
(a)



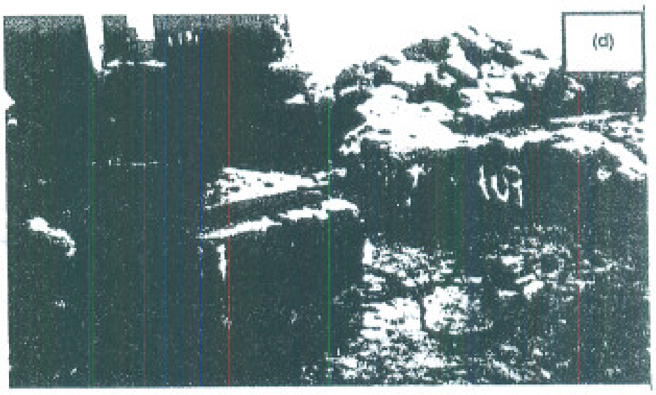
(b)



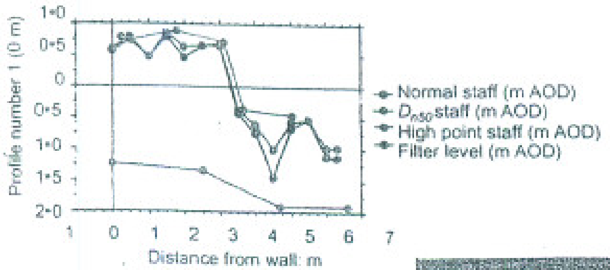

(c)



(d)



(e)




(f)

| | Bee | Rec | Bee | Rec |
|---|---|---|---|---|
| W50 | 6·0t | 2·0t | 6·0t | 2·0t |
| No. of profiles | 4 | 13 | 4 | 13 |
| Survey method | $k_t$ | $k_t$ | $n_v$ | $n_v$ |
| High spot | 0·92 | 0·98 | 36·8 | 38·2 |
| $0{\cdot}5D_{n50}$ foot | 0·85 | 0·94 | 31·3 | 34·4 |
| Normal staff | 0·79 | 0·85 | 26·2 | 27·0 |

Fig. 6. The Reculver study (see text for details)

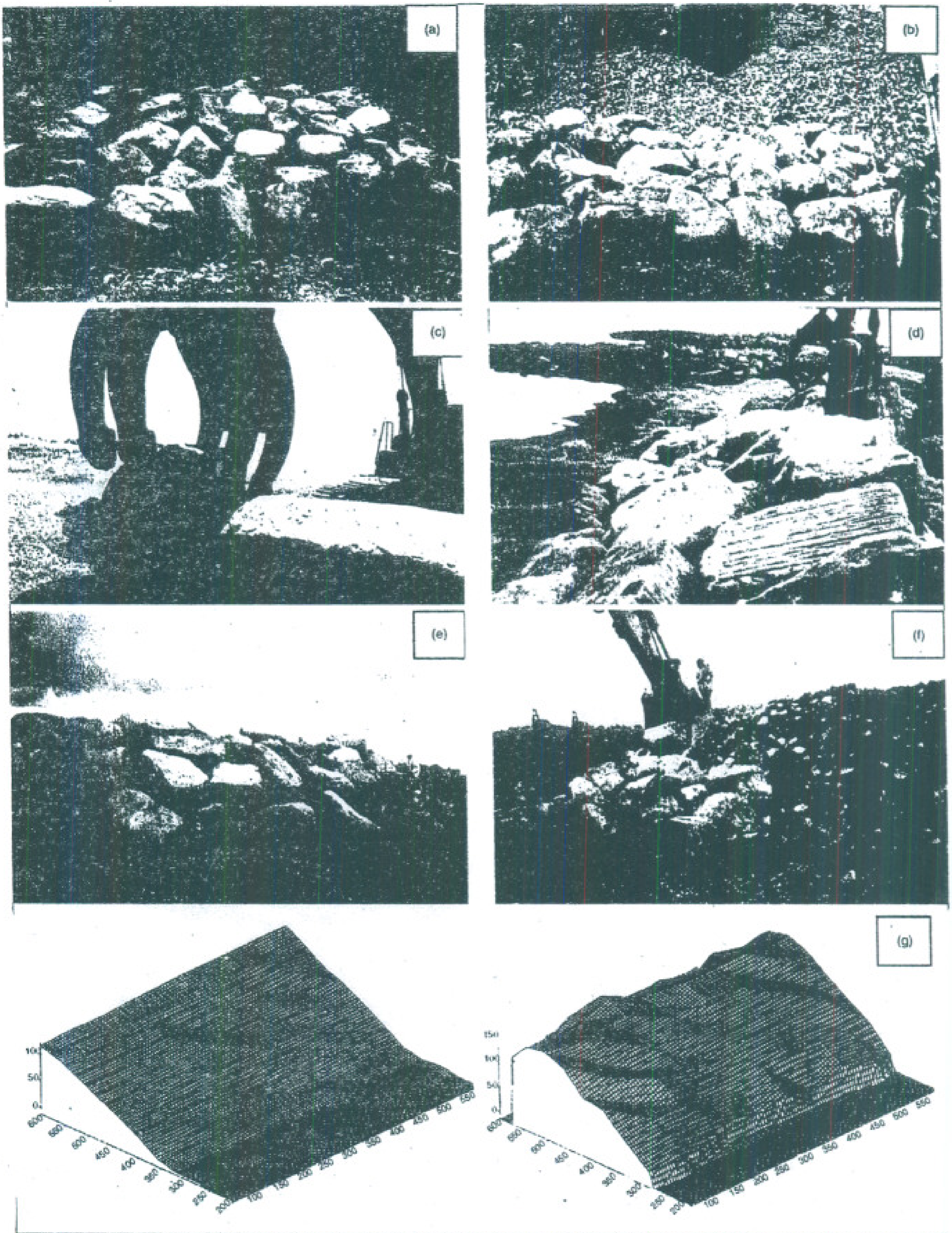



Fig. 7. Full-scale experimental revetments built during the DETR/HR/IC study (see text for details)

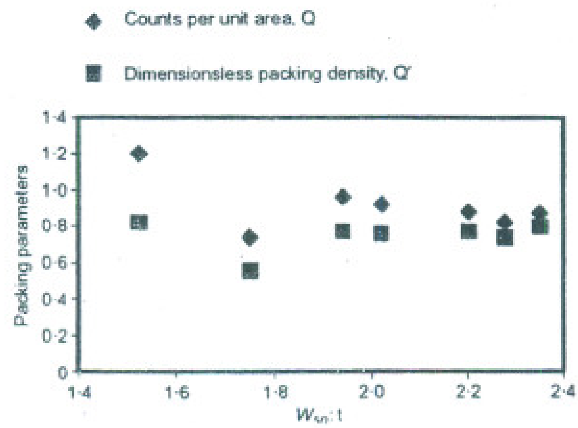



Fig. 8. Packing deduced from block count surveys. Comparison of seven different panels from a series of adjacent rock groynes and a revetment in south-east England, with the same $W_{50}$ and build specification

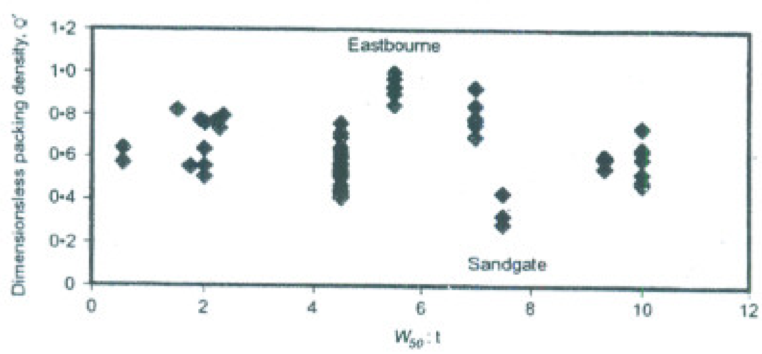



Fig. 9. Packing density $Q'$ from block counts assessed on 74 panels from 12 projects built with Larvik armourstone

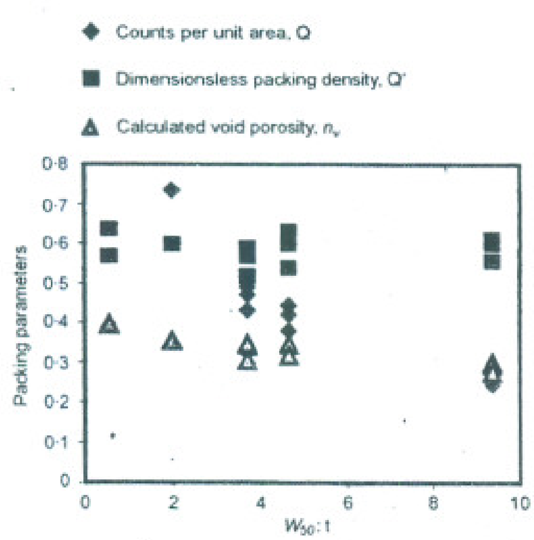



Fig. 10. Packing parameters deduced from block counts on 12 DETR/HR/IC study panels

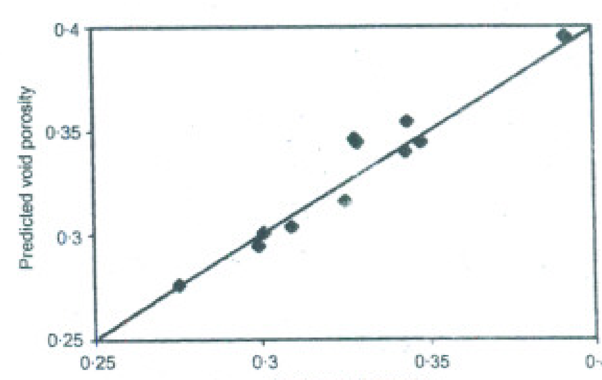



Fig. 11. Validation of void porosity prediction given in equation (5) using block count surveys of DETR/HR/IC study panels

# MATHEMATICAL MODELLING OF ALLUVIAL RIVERS : REALITY AND MYTH.
# PART I : GENERAL REVIEW

By

**Z. Cao and P. A. Carling**

Mathematical modelling fluvial flow, sediment transport and morphological evolution started half a century ago and, to date, a variety of mathematical models have been developed and are in widespread use. However, the quality of mathematical river modeling remains uncertain because of : (a) poor assumptions in model formulations ; (b) simplified numerical solution procedure ; (c) the implementation of sediment relationships of questionable validity ; and (d) the problematic use of model calibration and verification as assertions of model veracity. An overview of mathematical models for alluvial rivers is provided in this and the companion paper 'Part 2 : Special issues'. This paper is the first part, providing a general review of mathematical river models. The issues addressed comprise what have been obvious since the very beginning of mathematical river modeling and are still open, and also the pertinent components that pose challenges to model developers and end-users pursuing refined modelling practice. In particular the simplified mass conservation equations, asynchronous solution procedures, sediment transport functions, movable-bed resistance, numerical difficulty for strong hyperbolic equations, and representation of movable and complex geometry are discussed. A test example is provided to demonstrate the impacts of simplified mass conservation equations and an asynchronous solution procedure in comparison with those of largely tuned friction factors. It is concluded that mathematical models for fluvial flow sediment-morphology systems are far from being mature, and that considerable expertise, physical insight and experience are vital for meaningful solutions to be acquired and for the limitations of modelling outputs to be properly identified, interpreted and assessed.

## NOTATION

| | |
|---|---|
| $C$ | mean volumetric sediment concentration |
| $f$ | Darcy-Weisbach friction factor |
| $f_0$ | Darcy-Weisbach friction factor under initial condition |
| $h$ | flow depth |
| $L_1$ | normalised $L_1$-norm between changes of bed elevation |
| $R_T$ | relative time-scale of bed deformation |
| $t$ | time |
| $U$ | mean velocity |
| $w$ | sediment settling velocity |
| $x$ | streamwise coordinate |
| $Y_{nor}$ | normalised change of bed elevation |
| $\Delta Y$ | change of bed elevation |

## 1. INTRODUCTION

The ability to make accurate calculations of fluvial flow, sediment transport, the associated morphological evolution processes and water quality is vital in a period when the concern over the river environment and the influence of human intervention is increasing. The interaction between sediment and turbulent flow is of fundamental interest in the field of two-phase flow, and modelling the strongly coupled flow sediment-morphology system provides a problem of considerable interest in computational fluid dynamics. Fluvial sediment transport process has been an increasingly important subject in the fields of water resources engineering, hydrology, geographical, geological, and environmental sciences, and more fundamentally fluid dynamics.

The last half a century has seen encouraging progress in the development of physically based mathematical models relevant to such processes due to the advancement in the performance of computers, as can be found in a huge number of publications in the literature. There has been a significant move from comparatively simpler cross-section-averaged one-dimensional (1D) models in the early 1950s and depth-averaged two-dimensional (2D) models since the 1980s to full three-dimensional (3D) models more recently. To date, mathematical river modeling has become one of the basic tools for engineering planning, design and assessment. It has, since its early stage of development, been evolving towards one of the proactive problem-solving technologies for the river environment.

Refined mathematical modelling of alluvial rivers continues to be one of the major challenges for river scientists and engineers. Because the knowledge of the physics of turbulent flow, sediment transport, and their manual interaction is far from complete, existing mathematical models have inevitably come with ambiguity due to the underlying assumptions, approximations and, from time to time, ignorance. In contrast to the likely accepted premise that computational hydraulics for fix-bed rivers, based on the traditional Saint-Venant equations, matured prior to 1960, that for movable-bed rivers with sediment transport and morphological developments remains premature. River scientist and engineers do not have full confidence in making reliable and accurate simulations of sediment transport, whilst the users' community is moving towards a position where rapid impact-modelling and decision-making are required with decision support models and hydroinformatics tools. Identifying and delimiting the critical issues will lead in time to the development of improved models for alluvial river processes, which will offer the opportunity for better assessment of project risk.

The primary aim of this and the companion paper is to provide an overview of mathematical models for alluvial rivers. It is also intended to spur a wider awareness of the various uncertainties inherent in such models. This paper is the first part of the overview. Firstly a few basic aspects regarding the idealised steady and uniform sediment-carrying flow are highlighted to indicate the limited capability in quantitative description of sediment-laden flows. Then the major issues of mathematical river models are listed, and addressed. A test example is provided to illustrate the considerable impacts of the asynchronous solution procedure and simplified continuity equations. In the companion paper, several special issues are recalled in detail, which pose the basic impediments to refined mathematical river modelling and merit further fundamental research. These include turbulence closure models with particular emphasis on the role of sediment in modifying turbulence, the bottom boundary conditions for mean flow and sediment, and the calibration and verification/validation methodology widely used in mathematical river modelling. Suggestions are also made that can improve the current mathematical river modelling practice.

## 2. THE REALITY WITH IDEALISED STEADY AND UNIFORM FLOW

An idealised case is steady and longitudinally uniform water-sediment flow, which is mathematically the simplest to solve. Currently it is impossible to predict this kind of flow with sufficient confidence. Consider the following basic realities.

(a) It is difficult to correctly compute the mean velocity structure along the vertical. Often it is necessary to resort to a modified von Karman coefficient, Prandtl's mixing theory, or the weakly stable stratified flow analogy that was initially proposed for atmospheric boundary layers.

(b) It is difficult to correctly compute the vertical profile of mean suspended sediment concentration. From time to time it is required to tune, for instance, the Rouse parameter to fit concentration profiles to measured data. There is no universal guidance for use in tuning the Rouse parameter.

(c) It is difficult to accurately determine the mean flux of sediment exchange between the water column and the bottom sediment surface. It is necessary to use plainly questionable empirical relationships for the flux of bed sediment exchange in order to estimate the sediment discharge carried by the bulk flow.

Given the above statement for the idealised sediment-laden flow, it is undoubtedly of critical significance to recognise the basis, capabilities and limitations of current mathematical river models for the generally more complex fluvial water flow-sediment-morphology systems.

## 3. MAJOR ISSUES OF MATHEMATICAL MODELS FOR ALLUVIAL RIVERS

Mathematical models of alluvial rivers can be categorised into two types : academic and applied. Academic models often deal with 'how and why' problems, being devoted to the conceptualisation, mathematical formulation, solution (analytical or numerical) and interpretation of the flow, sediment transport, and morphological reaction. Improving the understanding of the mechanism of interaction among water, sediment and morphology is the major purpose of academic models. On the other hand, applied models are entirely concerned with quantitative modelling of the river systems in response to natural changes (such as climate change) and human activities (e.g. construction of dams, bridges and flood control works). The present overview considers applied modelling only. Currently, the most extensively used fluvial models are either 1D or depth-averaged (shallow) 2D, which are built upon traditional hydraulics principles – that is, Sanit-Venant equations. In the present overview the 1D and 2D models are distinguished from full 3D models based on the complete fluid dynamics equations implemented with a closure module for turbulence. Additionally, consideration is limited to issues directly related to sediment movement. Those issues purely pertinent to single-phase water flows (without sediment) in rivers are only briefly covered, which however must not be construed as irrelevant.

Table 1 lists a number of major issues of applied mathematical models for alluvial rivers that are addressed, and how these are relevant to various models. These issues encompass a variety of perspectives in fluid dynamics, sediment transport mechanics, morphological consideration, computational aspects as well as controversy pertaining to the methodology of model assessment, etc. In Table 1 the various issues are grouped according to these perspectives which are physically related to a wide spectrum of temporal and spatial scales, ranging from turbulence micro-scales to river-reach scales. In the following, several closely interrelated issues are discussed together.

## 4. 1D AND 2D COMPUTATIONAL HYDRAULICS MODELS

This section mainly focuses on 1D hydraulic models, while most aspects examined here are pertinent to depth-averaged 2D cases. Based on cross-section-averaged variables, 1D numerical modelling of alluvial rivers has been most widely used in the fields of rivers training, hydropower generation, flood control and disaster alleviation, water supply, navigation improvement, as well as environment enhancement. HEC-6, ISIS-Sediment, and Mike11 are examples of a number of mathematical river models developed for fluvial water-sediment-morphology systems. The outputs of these models usually include sediment transport rates, changes in bed elevation and amounts of erosion and deposition throughout the river system considered. It has been recognised that 1D models are appropriate primarily for long-term and long-reach situations, whereas these models have been less successful for local flow-sediment-morphology problems as can be anticipated. Prior studies in this connection have focused on such aspects as flow resistance relations (including parameter identification and optimisation), grain sorting, non-equilibrium modules, numerical techniques, and effects of vertical distributions. In the present state-of-the-art, it is a common practice to tune the friction factor and sediment transport formulae to reconcile the computational results with measurements. In this section the fundamental components of 1D models are examined. In particular the effects of simplified continuity equations and the asynchronous solution procedure are addressed, which have rarely been studied before except for a formative comparison.

**Table 1. Major issues and their relevance in applied mathematical models for alluvial rivers**

| | Issues | Relevance in models (Y = yes, N = no) 1D and 2D models | 3D models |
|---|---|---|---|
| Fluid mechanics | Saint-Venant equations | Y | N |
| | Reynolds-averaged Navier-Stokes equations | N | Y |
| | Resistance/roughness | Y | Y |
| | Turbulence closure future models and sediment effects | Y | Y |
| Sediment transport mechanics | Equilibrium versus non-equilibrium sediment transport | Y | Y |
| | Sediment transport and entrainment functions | Y | Y |
| | Heterogeneous sediments | Y | Y |
| | Temporal and spatial lags of bed-load transport | Y | Y |
| | Sediment exchange with bottom boundary | Y | Y |
| Morphological considerations | Complex river geometry representation | Y | Y |
| | Riverbed mobility implementation | Y | Y |
| | River bank migration | Y | Y |
| Computational aspects | Synchronous versus asynchronous solution procedures | Y | Y |
| | Hyperbolic equations | Y | N |
| Model assessment | Calibration | Y | Y |
| | Verification/validation | Y | Y |

### 4.1. Simplified continuity equation for water-sediment mixture

Alluvial flows over erodible beds can be distinguished from those over fixed beds in that the flow may entrain sediment from the bed or in contrast render the sediment carried by the flow to be deposited on the bed, which usually causes riverbed degradation or aggradation. This aspect is referred to as the bottom mobile (free) boundary problem. At the same time, the water-sediment mixture may have properties (density, etc.) different from clear water. In spite of these apparently known features of erodible-bed alluvial flows, it is often assumed that the rate of bed morphological evolution is of a lower order of magnitude than flow changes with adequately low sediment concentration. Accordingly, in existing 1D and 2D models, the water-sediment mixture continuity equation is almost exclusively assumed to be identical to that for a clear-water flow over a fixed bed without considering the alluvial riverbed mobility. This simplified mixture continuity equation is, in its from, the same as that in the traditional Saint-Venant equations. The effect of this treatment appears to have been quantitatively addressed only by Correia and discussed by Rahuel. Stevens claimed that bed mobility is important for complete coupling of water and sediment in discussing Lyn's analysis. Wormleaten and Ghumman in his paper compared the performance of several simplified models, but exclusive of a fully coupled model on a rigorous basis. Therefore the effect of bed mobility on model performance has not been apparent.

### 4.2. Simplified continuity equation for global sediment material

In addition to the widely invoked assumption with respect to the continuity equation for water-sediment mixture, the global bed material continuity equation (hereafter referred to as 'sediment continuity equation') is sometimes simplified by ignoring the temporal change related to sediment discharge or concentration i.e. sediment storage in the water column. Similarly in non-equilibrium models in which bed-load and suspended load are separately considered, the temporal variation associated with bed-load discharge is not accounted for. In depth-averaged 2D models, the relevant term is almost exclusively ignored. In reviewing existing models for river width adjustment, the ASCE Task Committee state that 'spatial differences in sediment flux . . . determine the evolution of the bed topography via solution of the sediment continuity equation'. Actually, the temporal change in relation to sediment discharge is disregarded in the sediment continuity equation. The validity of this simplification in the sediment continuity equation is still far from clear.

### 4.3. Simplified equations in analytical models

It is interesting to note that there have been several analytical models for channel aggradation and degradation. Whereas providing an easy-to-use approach to evaluating the response of channels to the changing of a simple water and sediment hydrograph or base lowering, these models are based heavily on assumptions. First, the flow is assumed to be quasi-steady, leading to the elimination of local derivatives in the water-sediment mixture continuity and momentum equations. Second, in the momentum equation the nonlinear convective acceleration term is ignored, yielding a diffusion model for bed elevation evolution. A slightly modified type of models, namely hyperbolic models have been developed by partly including the non-linear convective effect using a perturbation technique.

Finally in the sediment continuity equation the temporal concentration term is almost exclusively not taken into account in order to make the analytical solution tractable. One of the major difficulties in using these analytical models is the determination of the model coefficients involved. Additionally, it appears not encouraging to use these analytical models with highly variable hydrographs (complicated boundary conditions).

It is necessary to recognize that the momentum equation for the mixture flow over erodible bed differs from that of fixed-bed clear water flow. However, it seems a common practice to reduce it to a clear water flow momentum equation, recognising the uncertainty inherent in the resistance relationship that must be incorporated to close the momentum equation.

### 4.4. Equilibrium versus non-equilibrium models

In equilibrium models sediment transport is assumed to be at its capacity that is prescribed by one of a range of sediment transport functions involving local hydraulic parameters and sediment properties. Often the lumped total-load concept is used without discriminating the physically distinct mechanisms of bed-load and suspended load movements. Obviously the exchange between suspended sediment, bed-load and bed surface material cannot be explicitly represented in equilibrium models. Non-equilibrium models are at least intuitively more advanced than equilibrium models because they account for the limited availability of sediment under some special conditions, and more notably the time and space for sediment transport to adapt to its possible capacity in line with the local flow scenario. Earlier 1D models are mostly equilibrium models and recent ones appear to be non-equilibrium. Since the 1980s, the majority of models developed by Chinese workers are non-equilibrium. Nevertheless, there has been no study of the methodology whereby non-equilibrium models can be convincingly confirmed to be superior to equilibrium ones because of the uncertainty associated with both types of models.

One of the major sources of uncertainty with equilibrium models comes with the sediment transport function that must be introduced to determine sediment transport rate or discharge. As far as non-equilibrium models are concerned this feature is reflected by the relationships that must be incorporated to determine the net flux of sediment exchange with bed material. Detailed descriptions follow.

### 4.5. Sediment transport functions

To close an equilibrium model, a function is necessary to determine sediment transport rate and, for heterogeneous sediments, the size distribution of bed material being transported. A large number of functions have been developed. However, most, if not all, of these functions have been confirmed using specific laboratory and/or field measurement datasets, and none has been proved to be universally correct. Also it cannot be stated which function is the 'best' to use for a given situation. Distinct sediment transport functions will yield different answers, and normally the sediment rates/discharges are more sensitive to the choice of sediment function than the changes of river morphology. The latter concurs with the known feature that the time-scale of changes in flow variables (velocity, depth and sediment discharge, etc.) is normally significantly less than that of bed evolution. This aspect will be recalled later with respect to the asynchronous solution procedure community used in current mathematical river modelling practice. Therefore model developers and end-users have to judge the computational results based on their experience and their understanding of the basis on which existing sediment transport functions were derived and validated. Undoubtedly the modelling output is still subject to model developers and end-users-the lack of objectiveness is apparent.

Using both laboratory and river datasets, Yang and Wan compared the performance of several sediment transport functions that are popularly used, and showed that, for river datasets considered, the accuracy in ascending order was Engelund-Hansen, Laursen, Colby, Ackers-White, Einstein, Toffaletti, and Yang. At the same time Yang and Wan claimed that the rating does not guarantee that any specific function is better than others under all hydraulic and sediment cases. For gravel-bed rivers, the formulae of Einstein, Parker, and Ackers-White were shown to perform reasonably well. To measure the applicability of sand transport functions, an 'applicability index' was proposed by Williams and Julien on the basis of river characteristics. These authors argue that developing a universal (at least to a certain extent) procedure to help choose the 'optimum' sediment transport function among the large pool of candidates will be one of the most realistic strategies, to cope with the uncertainty due to sediment transport functions.

### 4.6. Model for bed-load transport

In a number of mathematical river models, bed-load sediment is modelled in a distinct module involving a transport formula (or function) in supplement to the module for suspended sediment. The total sediment transport rate and the global change in bed morphology are computed by summarising the contributions respectively of bed-load and suspended load. It is a commonly accepted belief that the accuracy of existing bed-load formulae is dramatically lower than that of suspended load relationships. Further, this is aggravated by the knowingly spatial and temporal lag of bed-load transport with respect to the change of flow conditions, which renders bed-load transport to be non-equilibrium. This induces much less confidence in mathematical models when bed-load transport dominates. Although a few sporadic formulations have been developed and implemented into 1D and even 3D models, their validity has only been occasionally shown when applied to specific problems. Furthermore, modelling bed-load sediment transport is made more complicated due to grain sorting of heterogeneous sediments. This is often dealt with based on the concept referred to as 'active layer'. One may refer to Parker for a brief description and some new derivations and Pender for a flume experimental analysis. An objective assessment of these formulations remains to be made. This status is certainly correct for models of arbitrary spatial dimensions.

### 4.7. Net flux of sediment exchange

A pivotal aspect of non-equilibrium models is the determination of the net flux of sediment exchange between the water column and the bed surface. The net flux of sediment exchange with bed material is the difference between the upward entrainment flux due to turbulence and the downward deposition flux under the gravitational action. It is normally defined at a specified reference elevation near the bed. Determining the net flux of sediment exchange is virtually generic to any spatially dimensional models for fluvial sediment transport. In 1D and depth-averaged 2D models, this comes as the closure of the source-sink terms in the mass conservation (or continuity) equation for sediment, whilst it is manifested as specifying the bottom sediment conditions in vertical 2D and 3D models. Given its generic feature, it is reserved as one of the special issues.

### 4.8. Resistance relation

A relationship for hydraulic resistance must be incorporated to close the momentum equation in 1D and 2D models. This involves another important topic in fluvial hydraulics, which has been the theme of a number of studies. Recent analysis can be found in Karim, Wu and Wang and Rooseboom and Grange.

The complexity of fluvial river resistance stems not only from the irregular boundary but the sediments carried by the flow. Since Vanoni's study, there has been much controversy concerning the effects of sediment on flow resistance (increase or decrease). The earlier works are largely concerned with the idealised steady and longitudinally uniform flow. Vanoni proposed that flow resistance decreases due to suspended sediment, based on the increase of the mean velocity gradient with sediment concentration along with the reduced von Karman coefficient. On the contrary, Elata and Ippen, Daily and Roberts as well as Montes and Ippen suggested that flow resistance increases because of either neutrally buoyant particles at high concentrations or natural sands of low concentrations. The recent study by Lyn shows that the flow resistance increases due to dilute suspension of sands.

The effects of suspended sediments on turbulent flow have been studied widely in China by Zhang R. and Xie J. and Chien N. and Wan Z. The sediments studied are mostly natural fine sand and clay (typically 0.1 mm in medium diameter or finer) at naturally occurring low and high concentrations (up to 0.5 in volume). The results mostly support the contention that flow resistance decreases, resulting from the suppression of turbulence by suspended sediments. The formulation of suspended sediment transport capacity based on this concept has been incorporated in various analytical and mathematical models.

Starting from the law of the wake in the turbulent boundary layer, Coleman argues that the mean velocity follows the logarithmic profile as in a clear-water flow in the near-bed region, and deviates from that according to the law of the wake near the free surface. The wake intensity varies as a function of sediment concentration. This 'universal' velocity profile is used by Lau, which inevitably leads to a decreased flow resistance. The novel framework of Lyn, based on dimensional and matching arguments, requires much empirical input. In its present form, it seems unable to properly represent the effects of suspended sediment on flow resistance.

A more popular approach for suspended sediment-laden flow is based on the wall-bounded slightly stably stratified flow analogy. The gradient Richardson number or, alternatively, the Monin-Obukhov length scale is exclusively invoked, both of which are commonly used for constant flux atmospheric boundary layer. This hierarchy of approaches always results in a reduction of flow resistance. It is necessary to recognise that the validity of this simple analogy is not clear or strictly justified. There exists substantial variability of the model parameters. McLean suggests a systematic model for sediment-laden flows using a universal von Karman constant. He finds that the inclusion of stratification effects is necessary but fails to specify accurately the constants that are needed in the parameterisation of the stratification effects. This, as a matter of fact, arises from the lack of existence of physically universal parameters in the stratified flow analogy-based model for sediment-laden flow.

It continues to be difficult to pinpoint the friction factor, especially of natural rivers. Often the Darcy-Weisbach frication factor ( $f$ ), alternatively the Manning roughness, has to be tuned to reconcile the mathematical modelling outputs with measurements. The uncertainty of this practice does merit clear recognition when assessing model performance. This holds true for 3D models addressed below, except that it is reflected by the boundary hydrodynamic roughness (e.g. $z_0$ in the literature) that is invoked in the law of the wall for bottom boundary conditions.

### 4.9. Asynchronous solution procedure, quasi-steady flow, and fixed-bed assumptions

The fluvial water-sediment-morphology system is strongly coupled, as clearly demonstrated in the governing equations of mathematical models. The physical mechanism of the interaction between the various components in the coupled system underlying these formulated equations has been well interpreted in text-books. In existing models, these equations are mostly solved in an asynchronous procedure. Specifically, in a given time step, the mixture continuity and momentum equations are solved first, assuming negligible bed change rate (i.e. fixed morphology). Then the sediment continuity equation is solved using the flow variables newly obtained. Models involving the asynchronous solution are usually referred to as decoupled. There have been semi-coupled models in which the flow and bed equations are solved iteratively in a given time step, yet the computational cost seems comparable or even higher than the fully coupled models that simultaneously solve the complete set of governing equations. It has been claimed that semi-coupled models permit arbitrary sediment discharge formulae to be easily incorporated. As a matter of fact, fully coupled models also allow for an expedient use of arbitrary sediment discharge formulae.

The asynchronous solution procedure is based on the 'fixed bed' and 'quasi-steady flow' assumptions. Often the flow is assumed to be steady when the evolution of the riverbed is studied. Alternatively, the riverbed is implicitly assumed to be 'fixed' within a time step while the flow over the mobile bed is of primary interest. The validity of these assumptions is determined by the typical time-scales or relative magnitudes of the characteristic celerities corresponding to the free-surface flow and riverbed evolution respectively. De Vries analysed the relative celerities when the volumetric sediment concentration is negligible. Morris and Williams confirmed the results of De Vries and extended the analysis to cases with finite sediment concentrations. It has been found that water flow, sediment transport and riverbed evolution can be considered to be mathematically independent of each other only within very limited ranges of total-load concentration and Froude number. Beyond these ranges, the 'quasi-steady state' or 'fixed bed' assumption is no longer reasonable. Cao and Egashira further extended the analysis of Morris and Williams. They found that when sediment is transported predominantly in suspension or within limited ranges of bed-load (rather than total load) concentration and Froude number, the flow, sediment and riverbed can be solved asynchronously and thus the 'quasi-steady state' and 'fixed bed' assumptions are justified. Based on the Saint-Venant equations, Lyn identified the multiple time-scales of the flow-sediment-reverbed system. He showed that previous models, which reduce the number of conservation equations to be solved simultaneously from three to two under the 'quasi-steady state' or 'fixed bed' assumption, are unable to satisfy exactly either a general boundary condition or an arbitrary initial condition. And in situations with highly variable discharge and sediment inputs, the afore-mentioned assumption is not justified. Needham and his colleagues in their papers viz "Wave hierarchies in alluvial river flows" and "On non-linear simple waves in alluvial river flows : a theory of sediment bores" carried out a thorough analysis of the hyperbolic system for alluvial river flows based on simplified equations. Despite the studies stated above, the quantitative effect of the asynchronous solution procedure is not as yet fully understood.

Physically, riverbed deformation results from unbalanced sediment exchange between the water column and the bed surface. Two distinct processes are involved in sediment exchange-that is, bed sediment entrainment due to turbulence and sediment deposition due to gravitation. The rate of the sediment exchange can be scaled to wC (where w is the representative sediment setting velocity, and C is the mean volumetric sediment concentration). Thus the time-scale of bed deformation is $T_B$ = h/wC (where h is the flow depth). As the time-scale of the flow is $T_F$ = h/U (where *U* is the mean velocity), then one can define the relative time-scale of bed deformation as.

$$R_T = \frac{T_B}{T_F} = \frac{U}{wC} \tag{1}$$

By equation (1) it is indicated that the time-scale of bed deformation is generally larger than that of the flow because of low sediment concentration and setting velocity compared to the flow velocity. This is the basis on which decoupled modelling has been commonly used over decades. However, there has been the quantitative measure of $k_1$ for decoupling to be acceptable. It is interesting to consider the sediment when flood induced bed evolution process in the middle and lower Yellow River, China. Assume a typical volumetric sediment concentration of 0.1 (265 kg/m$^3$), a representative sediment particle diameter of 0.06 mm (with *w* = 0.226 cm/s), and flow velocity 1.0 m/s, then the relative time-scale of bed deformation is equal to 4424.8. A variety of decoupled river models have proven to fail when applied to the Yellow River flooding processes. This implies that in such cases $R_T$ of an order of magnitude of 4000, the synchronous solution should be pursued if the rapid bed evolution process is to be modelled correctly.

### 4.10. Numerical schemes for strong hyperbolic equations

The governing equations of 1D models for the fluvial flow-sediment-morphology system are generally hyperbolic. Perhaps in most cases the river flows are sub-critical and therefore the daily used numerical schemes are effective and efficient. However there do exist some natural flow situations for which specially developed numerical schemes must be devised to cope with the strong hyperbolic nature, for example, extreme flood-induced degradation and dam-break flow over erodible bed. In fact the mobile-bed dam-break flow problem has been insufficiently studied, albeit there has been an EU Concerted Action on Dam Break Modelling program, and also a great number of investigations of 1D and 2D numerical models for dam-break flows over fixed boundaries. There is scope for basic studies of advanced numerical schemes for the strong hyperbolic equations, which can be used for the mixed sub-and super-critical river flows, intense sediment transport and rapid morphological evolution. In this regard the complete continuity equations must be synchronously solved. The significance of this observation is shown in the test example that follows, as related to the aggradation process induced by sediment overloading.

It is well-known that for mobile-bed river models, the first flow-related and bed-related celerities are positive, and the second flow-related celerity is invariably negative, irrespective of the Froude number (either super-or-sub-critical flow) and sediment concentration. The latter is in sharp contrast to the fixed-bed cases, in which the second flow-related celerity is dependent on Froude number. Accordingly, for mobile-bed problems the influences from both upstream and downstream need to be properly considered. One the other hand, the numerical difficulties with the so-called 'mixed-flow' or 'transcritical flow' under fixed-bed situations may no longer exist, or at least be alleviated in mobile-bed cases. Seemingly decoupling the flow equations from the sediment continuity equation simplifies the solution procedure. However it is the artificial decoupling that gives rise to the need for extra and often cumbersome treatment to facilitate the boundary conditions and to ensure stability, including the special discretisation scheme to be adopted.

### 4.11. Coupled and decoupled models : a test example

A numerical example of river aggradations due to sediment overloading is given below. The behaviour of (a) simplified continuity equations for the water-sediment mixture and global bed material : and (b) the asynchronous solution procedure is addressed, in comparison with the role of tuned friction factor. Equilibrium models are used as an illustration, and the sediment transport function is derived from experiments. According to the governing equations and the numerical solution procedures used, the various types of models are summarised in Table 2. FCM denotes a fully coupled model that employs the complete equations and synchronous solution procedure. PCM means partially coupled model as the equations are simplified compared to FCM (but are solved simultaneously). DCM indicates decoupled model characterised by the asynchronous solution procedure utilised.

**Table 2. Summary of 1D models**

| Model | Governing equations | Numerical solution procedure |
|---|---|---|
| FCM | Complete | Synchronous |
| PCM1 | Bed mobility neglected | Synchronous |
| PCM2 | Sediment storage in water column neglected | Synchronous |
| DCM | Bed mobility neglected | Asynchronous |

The test case is concerned with the aggradations process due to sediment overloading. The flume dataset of Soni is referred to for quantitative comparison of the models. The initial condition was an equilibrium state under constant discharge and sediment load. Sediment overloading was activated by feeding additional sediment at the upstream end while water discharge remained unaltered. The case with the ratio of increased sediment discharge to initial equilibrium value being 4.0 is considered here. An earlier numerical modelling of the aggradations process can be found in Bhallamudi and Chaudhry.

To quantitatively measure the performance of a numerical model solution as compared to a reference FCM solution based on complete governing equations and synchronous solution procedure, the normalised change of bed elevation is defined as

$$Y_{nor} = \frac{AY\ (x.\ t)}{AY\ (x.\ t)_{REF}} \qquad (2)$$

For measurement of the overall difference between two model solutions, the normalized $L_1$ norm between changes of bed elevation is defined as.

$$L_1 = \frac{\sum_{i=1}^{N} [\text{abs}\ \Delta Y(x,t) - \Delta Y(x,t)_{REF}]}{\sum_{i=1}^{N} [\text{abs}\ \Delta Y(x,t)_{REF}]} \qquad (3)$$

Where x is the stream-wise coordinate, t is the time. $\Delta Y(x, t)$ is the change of bed elevation, the subscript 'REF' denotes the reference FCM solution, and N the total computational node number. A perfect agreement between two solutions corresponds to $Y_{nor} \equiv 1$, and $L_1 \equiv 0$.

Figure 1 shows normalised change of bed elevations of partially coupled models (PCM1 and PMC2). In running all the models, FCM, PCM1 and PCM2, the same value of friction factor is used ($Rf = f / f_0 = 0.8$, defined as the ratio of friction factor used to that under the initial equilibrium state). Within the time period shown, riverbed aggradations is significantly underestimated by PCM1 and PCM2 as the value of the normalised change of bed elevation is less than unity (Fig. 1), and the under-prediction by PCM1 or PCM2 becomes steadily more pronounced with increasing computational time due to error accumulation. For other experimental cases with less overloading the difference between FCM and PCM or PCM is less significant but still appreciable.

As interpreted in Cao and Egashira in their paper "Coupled mathematical modelling of alluvial rivers" a decoupled solution for this case is hampered by unavailability of an additional upstream boundary condition when $Fr > 1$ immediately after overloading is started. In other words, decoupled modelling is rendered mathematically ill-posed and unachievable, which is caused solely by the asynchronous solution procedure.

It is known that considerable uncertainty may arise due to the resistance relationship that must be implemented to close the momentum equation. Often the friction factor (alternatively Manning's coefficient) in a resistance relation is tuned to reconcile modelling output with measurements. Apparently, it is interesting to see if the impacts of the partially coupled and decoupled models (Table 2) are comparable with those of solely tuned friction factors, Fig. 2 shows the normalised $L_1$ norm between the changes of bed elevation for the present case. The influences of the partially coupled models increase steadily with time, being comparable with (greater than) those of tuned friction factors (FCM : $Rf = 0.72$ and 0.89) within (after) roughly 7.5 h of overloading. The significance of the use of complete continuity equations is thus evident for the calibration of models.

Natural alluvial rivers are subject to significant changes of water and sediment discharges with movable bed boundary. The rate of bed evolution can be high under some conditions. For instance, in the middle and lower Yellow River, China, it can be up to several meters within a few days in flooding seasons. In numerical models for such river processes, simplifying the mixture continuity equation (as for that over a fixed bed) and sediment continuity equation (neglecting sediment storage in the water column) can lead to substantial inaccuracy. This inaccuracy can be cumulative and become more pronounced progressively with increasing computational time. The asynchronous solution procedure either renders the physical process mathematically ill-posed in supercritical regimes or results in considerable errors. Further, these impacts are comparable with those of largely tuned friction factors, characterising their significance in calibrating numerical river models. Consequently, it is necessary to use the complete mixture and sediment continuity equations and also synchronous solution procedure for refined modelling of alluvial rivers. The use of a synchronous solution of simplified continuity equations is likely the major reason that a number of models have proved to fail for the routing of sediment-laden floods in the Yellow River.

## 5. 3D MATHEMATICAL RIVER MODELS

With the advancement of computer technology, 3D modelling has been becoming more and more attractive because of the very detailed information that can be acquired from these models in contrast to the comparatively simpler 1D and 2D hydraulics models. There is significant merit in the move towards 3D models, although substantial research is required to incorporate methods developed in other fields for dealing with boundary condition uncertainties. In an attempt to model the flow over and morphological evolution of dunes, Carling found that a (vertical) 2D model is unable to reproduce the flow and dune evolution process. This stems from the incapability of the vertical 2D model to reflect the spanwise (or transverse) variation of the flow structure, the unavailability of appropriate formulations for bed-load movement, and the sophisticated turbulence-sediment interaction over the dune. More broadly, for strongly localised flow-sediment-morphology problems (e.g. local scour around structures), mathematical modelling is still in its infancy, far from being of mature use in engineering practice.

### 5.1. RANS against DNS and LES

Currently direct numerical simulation (DNS) and large eddy simulation (LES) are only applicable to idealised flows with regular and fixed geometry and will not be of significant benefit to end-users in the foreseeable future. Full 3D modeling of turbulent flows are mostly built upon the Reynolds-Averaged Navier-Stokes (RANS) equations with the aid of a turbulence closure module. The complete Reynolds stress closure module involves a large number of partial differential equations to be solved and its superiority over eddy viscosity-based models (say, two-equation models) for mean flow quantities is, if ever, merely marginal or limited in most cases. It is also demanding in computing cost for any large-scale realistic river problems. Therefore, turbulent eddy viscosity-based closure modules continue to be most widely utilised and have seen considerable success in engineering practice. The turbulent closure is one of the key issues within the framework of full 3D river modelling.

### 5.2. Two-fluid, particle-tracking, and algebraic slip mixture models for suspended sediment

In light of the distinct treatment of suspended sediment particles in line with fluid flow, models for suspended sediment can be categorised into :

(a) two-fluid models

(b) particle-tracking models

(c) algebraic-slip mixture (ASM) models

In two-fluid models, both the fluid phase (water) and the dispersed sediment particle phase are respectively considered as a continuum, with each phase having its own continuity and momentum equations. The governing equations for both phases which are coupled through an inter-phase momentum exchange team. The interactions due to sediment particle-particle collisions, which are important for high concentration cases, are accounted for by semi-empirical constitutive relationships incorporated into the solid-phase momentum balance equation. Satisfactory performance of two models of this kind has been reported by Villaret and Davies for relatively high concentration flows. Nevertheless, it must be noted that :

(a) the continuum assumption for dispersed sediment phase is questionable, especially for low sediment concentration flows as is most natural rivers.

(b) the knowledge of district turbulence behaviour of water and sediment phases is fragmentary, which causes much more uncertainty in the methods of the turbulent correlations.

(c) the computing cost is high due to the momentum equations for sediment phase to be solved.

As far as the particle tracking models are concerned, sediment particles are tracked using the kinematic equation involving the fluid velocity field. A statistical analysis is warranted to acquire the time-dependent distribution of particles in terms of sediment concentration, discharge, etc. There have been some reports of this kind of models for idealised situations with regular flow boundaries, etc. The following concerns over the suitability of these models for suspended sediment in natural rivers should be recognized :

(a) the impractical computational cost, especially for fine sediment transport in large-scale natural rivers due to the prohibitively huge number of sediment particles to be tracked.

(b) the difficulty in implementing the particle-bed interaction due to irregular natural river boundaries.

(c) the particle-particle interactions that may need to be accounted for, particularly in the near-bed region.

The algebraic slip mixture models are probably the most widely used, in which the water-sediment mixture is considered as a continuum in the continuity and momentum equations. The sediment phase is calculated based on a separate continuity equation, within which the sediment velocity is computed from the mixture's velocity plus an algebraic relative slip velocity -- that is, sediment settling velocity due to gravitational action. The following context, where applicable, is devoted to issues associated with the ASM models.

**5.3. The bottom boundary conditions**

Within the framework of RANS for the water-sediment mixture flow, a turbulent closure model and the ASM for suspended sediment, one of the major issues is with specifying the bottom boundary conditions. In light of the physics of sediment transport in alluvial rivers, this is actually the core of the phenomenon. In the near-bed zone, turbulence production and dissipation occurs, and there is strong interaction between turbulence and sediment. In the context of mathematical modelling, this is manifested in the bottom boundary conditions for mean quantities. For instance with the K – ε turbulence closure model, bottom boundary conditions must be specified for the mean velocity components, turbulent kinetic energy and dissipation rate as well as the mean flux of sediment exchange, etc. Although the present overview is not intended to fully review the dynamics of sediment transport, it is necessary to recognise the basis on which the bottom boundary conditions are formulated, so that the corresponding uncertainty can be fairly appreciated. Due to its pivotal role in mathematical river modelling, this is reserved as one of the special issues, to be covered in the companion paper, "Mathematical Modelling of alluvial rivers : reality and myth".

### 5.4. Free surface and movable bottom boundary implementation

The free surface dynamic boundary condition provides that the normal stress and shear stresses are equal to zero (neglecting the vanishingly small viscosity of air and surface tension), while the free surface kinematic boundary condition states that a material point on the boundary remains on the boundary. The complete boundary conditions have been widely utilised in the field of computational fluid dynamics, including large eddy simulations. However, in the field of river hydraulics, the complete free surface conditions have not been rigorously incorporated. Conventionally, the free surface has been implemented using, for simplicity, hydrostatic pressure distribution with rigid-lid assumption, or based on the shallow water premise, which virtually renders the 3D flow field not fully uncovered. In this connection there is an evident ambiguity that remains to be assessed. Dating back to Hirt and Nichols, the strict VOF (volume of fluid) method has been widely implemented in flow codes. An alternative method to the free surface seems to be the (non-adaptive or adaptive) $\sigma$ - transformation, with which the rigorous kinematic and dynamic conditions are preserved. The proper implementation of the free surface can be indispensable when turbulent mixing near the free surface is to be modelled and that may influence fine suspended sediment transport.

As the interface between flow and riverbed, the bottom boundary of the flow may be movable because of the generally unbalanced entrainment and deposition fluxes of sediment. Its elevation is determined by the mass conservation (continuity) of sediment material. In the case of 1D and 2D modelling, this is represented by a term in the mass conservation equation for the water-sediment mixture. Its significance has been reviewed and shown in the previous section (Figs. 1 and 2). In 3D modelling a formulation of this continuity can be easily derived, which is in its form similar to the kinematic condition for the free surface. Nevertheless, the movable bottom boundary is unique in that the exchange of sediment across the bottom boundary continues to be one of the major challenges in fluvial sediment transport (as stated below). Moreover, the dynamic condition at the movable bottom boundary has been implemented, without exception, based on the law of the wall with respect to a fixed boundary. The validity of this treatment is far from clear, which is in contrast to that at the free surface where the shear and normal stresses are negligible in river flows.

### 5.5. Complex natural river plane geometry and bank migration

Generally, the plane river geometry is meshed using boundary-fitted co-ordinates. However, when riverbank erosion occurs as in many rivers (say, meandering rivers with erodible banks), the river width adjusts, which renders the plane geometry movable (time-dependent). A review of the mechanism and modelling of river width adjustment can be found in ASCE. If one continues to employ boundary-fitted co-ordinates based on the solution of partial differential equations (e.g. elliptic operator problems such as Poisson equations), the timely tracking of the movable plane geometry (river banks) undoubtedly entails much higher computing cost. The need is evident for more efficient representation of the irregular and movable plane geometry (perhaps an algebraic transformation-based representation would be a good choice). The modelling of riverbank erosion can be extremely intriguing. Recent work in the framework of 1D and 2D hydraulics modelling can be found in Darby, Darby and Thorne, Darby *et al.* and ASCE, which represent the current state of the art.

## 6. CONCLUSIONS

Mathematical models for alluvial rivers (flow-sediment-morphology systems) are at best imperfectly constructed, and at worst invalid. A number of crucial issues have been reviewed, for which solutions are still far in the future. Primarily these include the sediment transport and entrainment functions, resistance relationship (or hydrodynamic roughness) and turbulence closure model that must be implemented to close the governing equations and to specify the bottom boundary conditions. These issues become more acute when heterogeneous and bed-load sediment transport dominate. The numerical solution procedure (synchronous versus asynchronous) and discretisation schemes are the other major factors that bring extra uncertainty to mathematical river models, especially for strong hyperbolic problems.

The significance of these issues may vary from river to river, but for all existing mathematical river models the awareness of these issues is generally meager. These issues represent some of the most basic problems in fluid mechanics, sediment transport dynamics, and numerical mathematics. The authors do not wish the list in Table 1 to be construed as an appeal to include all variables related to these issues, which would otherwise make the models more complex. What should be incorporated in specific models must be examined in isolation. Normally more adjustable parameters render river models more flexible in reconciling modelling outputs to observations, but at the same time introduce additional uncertainty and thus less confidence.

The issues of sediment entrainment functions and turbulence closure models characterise the currently incomplete knowledge of the physics of fluvial sediment transport and constitute the core of the discipline in question. Also because of the uncertainties in mathematical river models, proper assessment of model performance (calibration verification or validation) is significant.

*(Courtesy : Water and Maritime Engineering – 154 Issue – 3)*

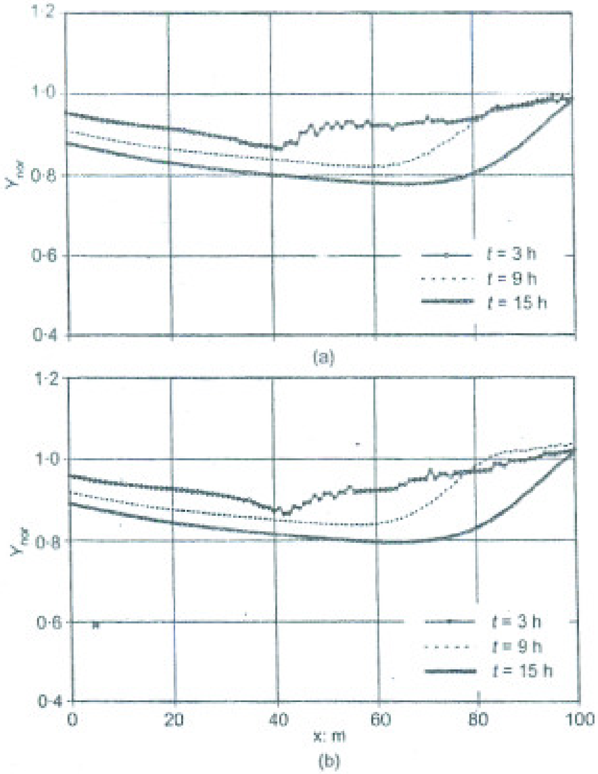



Fig. 1. Normalised change of bed elevations of: (a) PCM1; and (b) PCM2

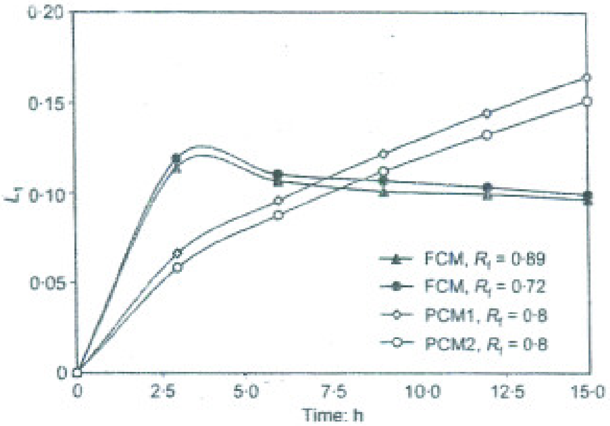



Fig. 2. Normalised $L_1$ norm between changes of bed elevation

# INFLUENCE OF PROJECT TYPE AND PROCUREMENT METHOD ON REWORK COSTS IN BUILDING CONSTRUCTION PROJECT

By

**Peter E. D. Love**
Associate Professor, School of Management
Information Systems, Edith Cowan University, Australia

## Abstract

While it is widely recognized that additional costs due to rework can have an adverse effect on project performance, limited empirical research has been done to investigate the influencing factors. The research presented in this paper aims to determine the influence of different project types and procurement methods on rework costs in construction projects. Using a questionnaire survey, rework costs were obtained from 161 Australian construction projects. The direct and indirect consequences of rework are analyzed and discussed. It is shown that, contrary to expectation, rework costs do not differ relative to project type or procurement method. In addition, it was found rework contributed to 52% of a project's cost growth and that 26% of the variance in cost growth was attributable to changes due to direct rework. To reduce rework costs and therefore improve project performance, it is posited that construction organizations begin to consider and measure them, so that an understanding of their magnitude can be captured, root causes identified, and effective prevention strategies implemented.

## Introduction

The doctrine of time is money appears to be well entrenched in the psyche of almost all construction industry clients. To satisfy the requirement of time (completion on time or earlier) a plethora of non-traditional procurement methods have emerged in the marketplace, which has resulted in design and construction schedules being compressed and construction commencing before the final design is complete (Hanna *et al.* 1999). As design and construction time is compressed, the degree of overlap (which will be referred to as concurrency) between activities increase, which in turn increases project complexity as activities are subdivided into trade packages. Hoedemaker et al, (1999) suggest that there is a limit to the number of tasks that can be undertaken in a concurrent manner. Beyond this specified limit, the probability of rework occurring as well as time and cost overruns being experienced significantly increases. Primarily this is due to the complexities associated with communication and co-ordination of a large number of tasks undertaken in a concurrent manner (Love *et al.* 2000a).

The problems associated with concurrency are exacerbated by time pressure, especially when rework occurs, and can accumulate as the data of project completion is approached. In an effort to meet the project's scheduled completion date, design and construction firms may have to employ additional resources. Such actions may lead to the opposite of the desired effect. That is, pushing beyond the limits of effective concurrency increases complexity, and this can increase the time to complete tasks (Brookes 1975). Considering these issues, nontraditional methods may in fact be subject to higher rework levels than traditional methods, especially when errors, omissions, and/or changes occur.

Moreover, under traditional methods, design and documentation are supposed to be complete, or largely complete, before construction commences on-site, so in theory there should be less rework attributable to design-related sources.

With this notion in mind, the primary objective of the research reported in this paper is to determine the influence different procurement methods and project types have on rework costs in projects. Using a questionnaire survey, rework costs from 161 Australian construction projects were obtained. From this survey, rework costs and their impact on project cost and schedule are analyzed and discussed. While the research was conducted in an Australian context, it is envisaged that the research outcome would be widely applicable in other locations. Prior to the presentation of the research, a brief review of the costs of rework in projects is presented.

**Rework Costs**

Various interpretations of rework can be found in the construction management literature. For example, terms such as quality deviations (Burati *et al.* 1992), nonconformances (Abdul-Rahman 1995), defects (Josephson and Hammarlund 1999), and quality failures (Barber *et al.* 2000) are often used, though these definitions vary. Ashford (1992) defines rework as "the process by which an item is made to conform to the original requirement by completion or correction". The Construction Industry Development Agency (1995), however, defined rework as "doing something at least one extra time due to nonconformance to requirements". Essentially, rework can result from errors, omissions, failures, damage, and change orders throughout the procurement process (Love *et al.* 1999 : Love and Li 2000b).

Josephson and Hammarlund (1999) reported that the costs of residential, industrial, and commercial building projects range from 2% to 6% of their contract values. Similarly, Love and Li (2000b) in their study of rework costs for a residential and industrial building found the costs of rework to be 3.15% and 2.40% of contract value respectively. In addition, Love and Li (2000b) found that when a contractor implemented a quality assurance system in conjunction with an effective continuous improvement strategy, rework costs were found to be less than 1% of the contract value.

The costs of quality deviations in civil and heavy industrial engineering projects, however, have been found to be significantly higher. Burati *et al.* (1992) studied nine major engineering projects to determine the cost associated with correcting deviations to meet specified requirements. The results of their study indicated that, for all nine projects, quality deviations accounted for an average of 12.4% of the contract value. A significantly lower figure was reported by Abdul-Rahman (1995), who found nonconformance costs (excluding material wastage and head office overheads) in a highway project to be 5% of the contract value. Abdul-Rahman (1995) specifically makes the point that the nonconformance costs may be significantly higher in projects where poor quality management is implemented. Notably, Nylén (1996) found that when poor quality management practices were implemented in a railway project, quality failures were found to be 10% of the contract value. Nylén (1996) further found that 10% of the quality failures experienced accounted for 90% of their total cost. Here significant proportions (76%) of the quality failure were attributable to design-related issues, such as erroneous documentation and poor communication between project team members.

As mentioned above, rework can also originate from change orders (Knocke 1993 : Love an Li 2000b). However, the extent to which change orders contribute to rework costs remains relatively unexplored. Research undertaken by Zeitoun and Oberlander (1993) found that the median costs of change orders for 71 fixedprice projects were 5.3% of the contract value and 6.8% for 35 cost reimbursable projects. Similarly, research undertaken by Cox *et al.* (1999) in the U. K. revealed that the costs of design-related change orders could range from 5% to 8% of the contract value, even when projects are managed effectively, as most of the changes are initiated by clients. The costs of change orders in the research reported by Zeitoun and Oberlander (1993) and Cox *et al.* (1999) are similar to the rework costs previously reported. A degree of change can be, and to a certain extent, should be expected in construction, as it is difficult for clients to visualize the end product that they procure. However, almost all forms of rework (with the exception of that caused by weather) are preventable, since poor management of the design and construction process typically causes such costs to occur.

**Research Methodology**

The research presented in this paper is part of a wider study that sought to determine the influence that project attributes, organizational management practices, and project management practices have on rework costs in construction projects. For the purposes of the research presented in this paper, rework is defined as 'the unnecessary effort of redoing a process or activity that was incorrectly implemented the first time'.

Rather than developing a questionnaire survey that sought respondents' general opinions about rework, the writer asked respondents to select a recently completed project most familiar to them and to subsequently answer questions about the perceived causes of rework, associated costs, and project management practices implemented. In essence, each project was treated as a separate case. In examining rework costs experienced in the projects sampled, the following two hypotheses were addressed :

1. There are significant differences in rework costs with varying procurement methods for construction projects ; and
2. There are significant differences in rework costs for varying types of construction projects.

***Questionnaire Survey***

Stratified random sampling was used to select the study sample from the telephone directory "Yellow Pages". In addition to increasing the representativeness of samples, stratified random sampling was a useful technique that made general statements about the portions of the population possible. Prior to determining the sample size for the main study, a pilot survey was undertaken with 30 selected firms, which comprised architects, project managers, and contractors from the Geelong and Melbourne region, in the state of Victoria. This was undertaken to test the potential response rate, suitability, and comprehensibility of the questionnaire. Each firm was contacted by telephone and informed of the aims of the research. On obtaining their consent, the questionnaire was mailed, with a stamped addressed return envelope enclosed, for respondents' returns, comments, feedback, and completion. The respondents were also asked to critically review the design and structure of the survey. All comments received were positive, and as a result the questionnaire remanied unaltered for the main survey. A total of 25 responses were received in the pilot survey, which equates to a response rate of 83%. The composition of respondents that returned the questionnaire was architects (30%), contractors (50%), and project managers (20%).

In the main survey, 70 questionnaires were mailed to each of the aforementioned categories of respondents throughout Australia, which equates to a total of 420 questionnaires distributed. One hundred thirty-six valid responses were received from the main survey. As the pilot questionnaires required no changes, they were added to the sample, which resulted in 161 valid responses representing a total consolidated response rate of 36%. This response rate is considered acceptable for a survey focusing on gaining responses from industry practitioners (Alreck and Settle 1985).

Fig. 1 provides a breakdown of the valid responses by respondent type. Contractors, architects, and consultant project managers accounted for approximately 81% of the respondents. While quantity surveyors (QSs) and structural, mechanical, and electrical engineers appear to be underrepresented ; it should be noted that many consultancy firms offer project management services and as a result may have undertaken a role of project manager for a project that they selected.

As the response rates from the QSs and the engineering profession were low, the respondents were recategorized under the following headings for analysis purposes :

1. Design consultants, which comprised architects, QSs, and structural, mechanical, and electrical engineers (44%) :
2. Contractors (33%) ; and
3. Consultant project managers (23%).

***Data Reliability***

Data reliability is related to data source and the identification of the position held by the person who completed the questionnaire (Oppenheim 1992). Thus, as it was important that the personnel who had detailed knowledge about the procurement processes associated with a project answered the questionnaire, it was mailed to the senior personnel within the organizations identified. Whether they actually answered the questionnaire was impossible to determine, except in circumstances where respondents optionally supplied their details at the end of the questionnaire.

From 161 questionnaires used in the research, 87 respondents provided their business details, and many held senior positions within their organizations. Based on the position titles of the respondents, the direct mailing to individuals in organizations seemed to have achieved its objective of reaching those who were closely involved with delivering construction projects. In addition, the questionnaires were mailed to organizations in different states in Australia, which minimized the duplication of selected projects.

Fig. 2 provides a breakdown by state of the respondents who answered the questionnaire. Considering the number of construction projects being undertaken in Australia at any given time, the likelihood of the respondents selecting the same project was significantly reduced because of the diversity of data sources from each state.

### *Method and Data Analysis Justification*

The respondents were asked to provide the following details :

1. Project and facility type,
2. Gross floor area (GFA),
3. Number of storeys,
4. Procurement method,
5. Tendering method,
6. Original contract value,
7. Contract value on practical completion,
8. Original construction period, and
9. Actual construction period.

In addition, estimates of direct and indirect rework costs that occurred during the project's construction were also sought. Procurement and tendering methods and project and facility types were measured on a nominal scale, while the other variables were measured on a ratio scale. Respondents were also asked to provide general comments at the end of the questionnaire with respect to the sources and causes of rework in their selected project. The data collected were analyzed using the Statistical Package for the Social Sciences for Windows, Version 9.00. Descriptive statistics were used to determine the central tendency and dispersion (mean and standard deviation) of the data. Such descriptive analysis provided a frame of reference for using appropriate inferential statistical techniques.

In order to test the association of the variable that were measured on a ratio scale with the rework costs, Pearson's parametric correlation was computed, and this approach enabled those variables with significant correlation at the 95% and 99% significance intervals to be identified. As well as determining the relationships between the variables, a one-way analysis of the variance (ANVOVA) was used to test for significant differences between rework costs for different procurement methods and project types. Moreover, to identify where any differences between samples lie, a Turkey's post hoc test was used.

## Analysis and Discussion

### *Project and Facility Type*

A variety of project and facility types and procurement and tendering methods were identified in the survey. Tables 1 and 2 present a summary of project and facility types and the respective procurement methods used to deliver projects obtained from the survey data.

Table 1 shows that traditional lump sum methods were typically used to procure new build (44.4%) and refurbishment projects (72%), whereas construction management methods tended to be used to procure fit-out projects (42.9%). New build project types accounted for 56% (90) of the total number of projects (Table 2). Refurbishment and renovation projects were found to be the second most popular type, accounting for 26% (43) of the total projects. Table 2 indicates that respondents were involved in the procurement of a variety of facility types. The most popular facility types were commercial offices (13.7%), residential buildings (13%), and hospitals/health (12.4%).

### *Procurement and Tendering Methods*

From the sample of 161 projects, the most popular procurement methods used to deliver project types were traditional lump sum (52.2%), design and build (19.9%), and construction management (16.1%). Previous research undertaken by Love *et al.* (1998b), which examined procurement selection, among clients and consultants, found that these forms of procurement methods were also predominant in the marketplace. Similarly, traditional lump sum methods are also the most popular forms of procurement in many other Commonwealth countries such as Malaysia (Hashim 1997), Hong Kong (Chan *et al.* 1999), Sigapore (Lam and Chan 1995), South Africa (Bowen *et al.* 1997) and the U. K. (RICS 1996).

**Table 1. Procurement Methods Used to Deliver Project Types**

| | Project type | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Procurement Method | New build | | Refurbishment/ renovation | | Fit-out | | New build/ refurbishment | | Combination of all | | Total | |
| | Number | (%) | Number | (%) | Number | (%) | Number | (%) | Number | (%) | Number | (%) |
| Traditional lump sum | 40 | 44.4 | 31 | 72.0 | 5 | 35.7 | 7 | 63.6 | 1 | 33.3 | 84 | 52.2 |
| Traditional cost plus | 1 | 1.1 | --- | ------ | --- | ------ | --- | ------ | --- | ------ | 1 | 0.6 |
| Traditional with provisional quantities | 5 | 5.6 | --- | ------ | --- | ------ | --- | ------ | --- | ------ | 5 | 3.1 |
| Design and manage | 1 | 1.1 | --- | ------ | 6 | 14.0 | --- | ------ | --- | ------ | 2 | 1.2 |
| Construction management | 11 | 12.2 | 6 | 14.0 | 6 | 42.9 | 3 | 27.3 | --- | ------ | 26 | 16.1 |
| Management contracting | 1 | 1.1 | --- | ------ | --- | ------ | --- | ------ | --- | ------ | 1 | 0.6 |
| Design and build | 22 | 24.4 | 6 | 14.0 | 2 | 14.3 | 1 | 9.1 | 1 | 33.3 | 32 | 19.9 |
| Novation | 5 | 5.6 | --- | ------ | --- | ------ | --- | ------ | 1 | 33.3 | 6 | 3.7 |
| Turnkey/package deal | 2 | 2.2 | --- | ------ | --- | ------ | --- | ------ | --- | ------ | 2 | 1.2 |
| Build-Own-Operate Transfer (BOOT) | 2 | 2.2 | --- | ------ | --- | ------ | --- | ------ | --- | ------ | 2 | 1.2 |
| Total | 90 | 100 | 43 | 100 | 14 | 100 | 11 | 100 | 3 | 100 | 161 | 100 |

Traditional methods of procurement have been heavily criticized for their sequential approach to project delivery, as they have contributed to the so-called "procurement gap" whereby design and construction processes are separated from one another (Love *et al.* 1998a). As a result, Love *et al.* (1998a) suggest that behavioural, cultural, and organizational differences between project individuals and groups often prevail. In addition, the procurement gap (between design and construction) is considered to inhabit communication, co-ordination, and integration among project team members and can adversely affect project performance (Lahdenperi 1995 : Exbuomwam and Anumba 1996).

**Table 2. Type of Facility and Project Procured**

| Project type / Facility type | New build Number | (%) | Refurbishment/ renovation Number | (%) | Fit-out Number | (%) | New build/ refurbishment Number | (%) | Combination of all Number | (%) | Total Number | (%) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Administrative authorities | 3 | 3.3 | 3 | 7.0 | 2 | 14.3 | --- | ------ | --- | ------ | 8 | 5 |
| Administrative civic | 8 | 8.9 | 2 | 4.7 | 2 | 14.3 | --- | ------ | --- | ------ | 12 | 7.5 |
| Banks | 1 | 1.1 | 2 | 4.7 | --- | ------ | 2 | 18.2 | --- | ------ | 5 | 3.1 |
| Educational school | 3 | 3.3 | 2 | 4.7 | --- | ------ | --- | ------ | --- | ------ | 5 | 3.1 |
| Educational university | 6 | 6.7 | 3 | 7.0 | --- | ------ | 2 | 18.2 | 1 | 33.3 | 12 | 7.5 |
| Entertainment | 5 | 5.6 | 1 | 2.3 | 1 | 7.1 | --- | ------ | --- | ------ | 7 | 4.3 |
| Hotel/motel/resort | 11 | 12.2 | 1 | 2.3 | 1 | 7.1 | --- | ------ | --- | ------ | 13 | 8.1 |
| Hospital/Health | 10 | 11.1 | 4 | 9.3 | 1 | 7.1 | 5 | 45.5 | --- | ------ | 20 | 12.4 |
| Commercial recreational | 9 | 10 | 2 | 4.7 | 1 | 7.1 | --- | ------ | --- | ------ | 12 | 7.5 |
| Commercial retail | 7 | 7.8 | 5 | 11.6 | 2 | 14.3 | 1 | 9.1 | 1 | 33.3 | 16 | 9.9 |
| Commercial offices | 8 | 8.9 | 9 | 20.9 | 3 | 21.4 | 1 | 9.1 | 1 | 33.3 | 22 | 13.7 |
| Industrial warehouses | --- | ------ | 1 | 2.3 | 1 | 7.1 | --- | ------ | --- | ------ | 2 | 1.2 |
| Industrial factory | 2 | 2.2 | 2 | 4.7 | --- | ------ | --- | ------ | --- | ------ | 4 | 2.5 |
| Residential | 16 | 17.8 | 5 | 11.6 | --- | ------ | --- | ------ | --- | ------ | 21 | 13 |
| Airport | 1 | 1.1 | 1 | 2.3 | --- | ------ | --- | ------ | --- | ------ | 2 | 1.2 |
| Total | 90 | 100 | 43 | 100 | 14 | 100 | 11 | 100 | 3 | 100 | 161 | 100 |

Nontraditional methods such as design and build and construction management have been advocated as methods for overcoming some of the problems inherent in traditional methods (NEDO 1988 ; Turner 1990 ; Masterman 1994), yet it would appear from these findings that their use is minimal (Table 2). Sharif and Morledge (1997) provide a plausible explanation for the ubiquitous use of traditional methods by stating that most construction clients are small and occasional and therefore only ever build once or twice. Thus, an architect is typically the first point of contact for these clients and their advice is heavily relied upon (Mackinder and Marvin 1982 ; Sharif and Morledge 1997). Consequently, it is often in the interest of the architect to persuade the client to use a traditional method, as they can take a lead role in the project as well as maximize his or her fees. Traditional methods can provide clients with cost certainty, whereas design and build and construct management methods are often used when the pressure of early completion is imposed on the project (Holt *et al.* 2000).

Table 3 indicates that single-stage tendering method (52%), then negotiation (32%), and finally two-stage tendering (16%). No other forms of tendering were identified as being used in the procurement of the 161 projects. Notably, single-stage tendering was predominantly used in conjunction with traditional methods, whereas negotioned types tended to be used more with nontraditional methods such as design and build and construction management procurement methods.

**Table 3. Tendering Procurement Methods Used to Deliver Project Types**

| Tender method / Procurement method | Single stage Number | (%) | Two-stage Number | (%) | Negotiated Number | (%) | Total Number | (%) |
|---|---|---|---|---|---|---|---|---|
| Traditional lump sum | 57 | 68.7 | 12 | 44 | 15 | 29.4 | 84 | 52.2 |
| Traditional cost plus | 1 | 1.2 | --- | ------ | --- | ------ | 1 | 0.6 |
| Traditional with provisional quantities | 1 | 1.2 | 1 | 3.7 | 3 | 5.9 | 5 | 3.1 |
| Design and manage | 2 | 2.4 | --- | ------ | --- | ------ | 2 | 1.2 |
| Construction management | 10 | 12 | 4 | 14.8 | 12 | 23.5 | 26 | 16.1 |
| Management contracting | --- | ------ | 1 | 3.7 | --- | ------ | 1 | 0.6 |
| Design and-build | 8 | 9.6 | 7 | 25.9 | 17 | 33.3 | 32 | 19.9 |
| Novation | 3 | 3.6 | 1 | 3.7 | 2 | 3.9 | 6 | 3.7 |
| Turnkey/package deal | --- | ------ | 1 | 3.7 | 1 | 2.0 | 2 | 1.2 |
| Build-Own-Operate Transfer (BOOT) | 1 | 1.2 | --- | ------ | 1 | 2.0 | 2 | 1.2 |
| Total | 83 | 100 | 27 | 100 | 51 | 100 | 161 | 100 |

In the U. K., however, the most popular methods of tendering, in order of popularity, have been found to be selective tendering, negotiation, and two-stage and open tendering (Holt *et al.* 2000). In an earlier study in the U. K., Bresnen *et al.* (1988) found that a quarter of contracts were negotiated, which is surprising in that negotiation removes the element of competition. Nonetheless, this correlates with the findings of the (then) Institute of Building (1979), who stated :

> Competition is not essential to achieve value for money. Evidence shows that contracts let by open or single stage selection appear less successful than those let by other means (whilst) negotiated contracts show less divergence between final, and tender values.

Whether there is actually less divergence between final and tender values when using negotiated forms of tendering will be explored using the ANOVA test. Thus, cost growth – the divergence between original and actual contract value on completion – was identified as the dependent variable and tendering type the independent variable. Levene's test of homogeneity of variances was not violated ($p < 0.05$), which indicates that the population variances for tendering method were equal. The ANOVA revealed no significant differences between each tendering method for cost growth, $F(2, 158) = 0.631$, $p < 0.05$.

Selective competition places emphasis on lowest cost, and when Bills of Quantities are included, cost certainty and quality can be achieved (Bresnen *et al.* 1988). However, according to Watkinson (1992), traditional methods tend to perform poorly in terms of time. In stark contrast, however, the findings reported above clearly indicate that there is no difference between procurement methods in terms of their schedule growth. Accordingly, Walker (1994) also found that procurement methods do not influence the time performance of projects.

***Project Size***

Fig. 3 identifies the number of projects by contract value. The contract values for the projects were found to range (in Australian dollars) from \$ A 25,500 to \$ A 390,000,000 (mean (M) = \$ A 25,521,927, standard deviation (SD) = \$ A 51,957,899). Likewise, the contract duration ranged from 2 weeks for a retail fit-out to 450 weeks for an airport terminal (M = 66 weeks, SD = 46 weeks). The gross floor area (GFA) for the 161 projects used in the analysis was found to range from 116 $m^2$ for a fit-out project with a contract value of \$ A 25,500 to 238,000 $m^2$ for a new build airport terminal with a contract value of \$ A 390,000,000 (M = 13,583 $m^2$, SD = 23,617 $m^2$).

Surprisingly, the fit-out project was estimated to have experienced rework costs of 12% (10% direct plus 2%indirect), whereas the airport terminal experienced only 2% rework costs (direct only). In determining whether there was a significant relationship between GFA and rework costs, Pearson's (r) correlation was computed (Table 4). The correlation for the data revealed that GFA and number of storeys were not significantly related to rework costs.

***Cost and Schedule Growth***

Construction projects are notorious for running over budget and time (Hester *et al.* 1991; Zeitoun and Oberlander 1993 ; Ibbs and Allen 1995). Change orders have been found to be a major contributor to time and cost overruns (Jahren and Ashe 1990), yet the impact that rework has on the cost and time performance of projects remains unexplored in the construction management literature. Hence, respondents were asked to provide the following details so that cost and schedule growth could be calculated for each project.

**Table 4. Correlation Matrix of Rework Costs and Project Characteristics**

| Project characteristics | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 |
|---|---|---|---|---|---|---|---|---|
| Cost growth | 1.00 | ------ | ------ | ------ | ------ | ------ | ------ | ------ |
| Gross floor area | 0.00 | 1.00 | ------ | ------ | ------ | ------ | ------ | ------ |
| Number of floors | 0.09 | 0.32$^{b}$ | 1.00 | ------ | ------ | ------ | ------ | ------ |
| Rework as percentage of cost growth | 0.00 | -0.12 | -0.12 | 1.00 | --- | ------ | ------ | ------ |
| Schedule growth | 0.31$^{b}$ | -0.12 | -0.08 | 0.15$^{a}$ | 1.00 | ------ | ------ | ------ |
| Total rework cost | 0.44$^{b}$ | -0.05 | 0.01 | 0.15$^{a}$ | 0.38$^{b}$ | 1.00 | ------ | ------ |
| Direct rework cost | 0.51$^{b}$ | -0.08 | -0.05 | 0.20$^{a}$ | 0.38$^{b}$ | 0.91$^{b}$ | 1.00 | ------ |
| Indirect cost of rework | 0.27$^{b}$ | -0.01 | 0.07 | 0.07 | 0.31$^{b}$ | 0.89$^{b}$ | 0.63$^{b}$ | 1.00 |

[a] Correlation is significant at 0.05 level (2-tailed).
[b] Correlation is significant at 0.01 level (2-tailed).

1. Original contract value,
2. Project's expected duration in weeks,
3. Actual contract value on practical completion, and
4. Actual construction period.

Cost and schedule growth for each project was calculated using the following formula (Zeiton and Oberlander 1993).

$$PROJECT_{CG} = \frac{\Sigma_{CVP} - \Sigma_{OCV}}{\Sigma_{OCV}} \qquad (1)$$

$$PROJECT_{SG} = \frac{\Sigma_{ACP} - \Sigma_{OCP}}{\Sigma_{OCP}} \qquad (2)$$

Where CG = percentage cost growth ; SG = percentage schedule growth ; CVP = contract value on practical completion ; OCV = original contract value ; ACP = actual construction period (weeks) ; and OCP = original construction period (weeks).

Table 5 displays the mean cost and schedule growth experienced for projects delivered using different procurement methods. Fig. 4 and Fig. 5 denote the aggregate distributions of cost (M =12.6%, SD = 24.22%) and schedule growth (M = 20.7%, SD = 28%) for the 161 projects. The maximum cost growth was 244% and the minimum – 84%, which results in a range of 328%. Moreover, the maximum schedule growth was 144% and the minimum – 29%, which results in a range of 170%.

It is evident from the findings presented above that cost and schedule overruns in projects were a common occurrence and the degree of overrun varied significantly among projects. Nevertheless, is there a difference between procurement methods for cost and schedule growths ? An ANOVA test was used in this instance to test for differences. Levene's test of homogeneity of variances was found not to be violated ($p < .50$), which indicates that the population variances for each group of the procurement methods for cost and schedule growth were equal. The ANOVA, however, revealed that there were also no significant differences between procurement methods and cost growth, $F(9,151) = 0.217$, $p < 0.05$. Nor is there any significant difference in schedule growths, $F(9,151) = 1.553$, $p < 0.05$.

**Table 5. Mean Schedule and Cost Growth for Different Procurements**

| Procurement method | Mean cost growth (%) | Mean schedule growth (%) |
|---|---|---|
| Traditional lump sum | 15 | 23 |
| Traditional cost plus | 19 | 67 |
| Traditional with provisional quantities | 13 | 10 |
| Design and manage | 5 | -12 |
| Construction management | 11 | 25 |
| Management contracting | -6 | -6 |
| Design and build | 11 | 13 |
| Novation | 8 | 34 |
| Turnkey/package deal | 9 | 8 |
| Build-Own-Operate Transfer (BOOT) | 9 | 2 |

The respondents were asked to estimate the direct cost of rework for the project they selected. The estimate provided by the respondents was then expressed as a ratio of cost growth so that the "rework as a percentage of cost growth" for each project could be determined. Fig. 6 displays a summary of direct "rework costs as a percentage of cost growth" (M = 52.1%, SD = 88.9%).

While the mean cost growth for the projects was found to be 12.6%, rework contributed to 52.1% of the cost growth, with factors such as weather, industrial relation, and client/end-user change orders contributing to the remaining 47.9%. Clearly, reductions in rework significantly improve the cost performance of projects. Noteworthy, however, these were cases that experienced high direct rework costs (> 10%) but came in under budget and experienced negative cost growths. In testing whether there was a significant relationship between cost growth and rework. Pearson's (r) correlation was computed (Table 4). The correlation coefficients for the data revealed that cost growth and direct rework were significantly related, $r = +0.51$, $n = 161$, $p < 0.01$, two tails and $r^2 = 0.26$ (26%). Similarly, indirect rework was also significantly related to cost growth, $r = +0.25$, $n = 161$, $p < 0.01$, two tails and $r^2 = 0.06$ (6%), through it should be noted that Hinkle *et al.* (1988) recommend that a correlation of 0.70 to 0.90 be considered high and 0.50 to 0.70 moderate. This "rule of thumb" was recommended because the r-value is used to derive the value for $r^2$, which indicates that one variable can be predicted by changes in another. Thus, 26% of the variance in cost growth can be attributable to changes due to direct rework.

The correlation analysis, presented in Table 4, also reveals that schedule growth and direct rework costs are significantly related, $r = +0.38$, $n = 161$, $p < 0.01$, two tails and $r^2 = 0.15$ (15%). However, the $r^2$ is low, which indicates that only 15% of the variance in scheduled growth can be predicted by changes due to direct rework. While this is a surprising finding, a detailed examination of the project data revealed that 27% of projects were delivered on or ahead of time despite experiencing cost increases due to rework. Once rework occurs, projects can be accelerated and resources reallocated to compensate for any delays that may arise, though this may be costly in dollar terms, especially if the rework influences the projects' critical path.

The occurrence of rework will invariably result in the contractors reevaluating their project schedules, as delays have the potential to lead to liquidated damages being incurred. If a delay occurs due to rework and the contractor is not responsible for it, then an extension of time or acceleration costs may be awarded, though this will depend on the type of delay and how it impacts the critical path. The reevaluation of a project schedule may well require project teams to work together to solve problems that may arise as a result of the rework. In doing so, they may identify alternative construction sequences and/or methods that can be used to complete the project on or ahead of time.

***Rework Costs***

The questionnaire survey asked the respondents to provide an estimate of the direct and indirect costs of rework that occurred in the project selected. Rework costs are very rarely, if ever, measured by Australian construction organizations (Love *et al.* 2000a), so that estimates provided by respondents were based on their cognizance of the project.

The total rework costs were calculated by adding the direct and indirect estimates provided by the respondents. Fig. 7 identifies the mean and standard deviation of total rework costs for the 161 construction projects (M = 12.0%, SD = 13.56). Figs. 8 and 9 display the distribution of the respondents estimates for direct (M = 6.4%, SD = 7.78%) and indirect (M = 5.6%, SD = 7.19%) rework costs. Though it was surprising to find that respondents considered indirect costs to be similar in amount to direct costs, especially as Love *et al.* (2000c) have shown indirect rework costs can have a cost-multiplier effect as much as five times the actual (direct) cost of rectification.

The total costs of rework have been found to vary considerably among projects. Some respondents reported rework costs to be less than 1% of a projects' original contract value, while others have reported them to be as high as 80%. The degree of variability in the estimates given by the respondents suggests that many respondents may be unsure about the actual costs of rework incurred in the projects. The mean estimates of practitioners' rework costs that were perceived to have been incurred in their selected projects are presented in Table 6.

To test whether there were significant differences between the estimates of the respondents for rework costs (direct and indirect) an ANOVA was undertaken. The descriptive statistics revealed differences between the design consultants (M = 8.0%, SD = 9.3), contractor (M = 5.8%, SD 7.2%), and project manager (M = 4.3%, SD = 3.9%) in their estimates of direct rework costs. Levene's test of homogeneity of variances was violated ($p < 0.05$), which indicates that the population variances for each respondent type were not equal. The ANOVA revealed significant differences between respondents' estimates of direct rework costs, $F(2,158) = 3.028$, $p < 0.05$. The results of Turkey's HSD post hoc test indicated that the differences identified in the estimates for direct rework costs were between design consultants and project managers ($p < 0.05$).

Table 6. Direct and Indirect Rework Cost Estimates by Respondents Type

| | | Direct rework costs | | | | | Indirect rework costs | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Respondent | N | Mean | Standard Deviation | Standard Error | Minimum | Maximum | Mean | Standard Deviation | Standard Error | Minimum | Maximum |
| Design consultant | 71 | 8.0 | 9.3 | 1.1 | 0.50 | 50.00 | 6.77 | 8.34 | 0.99 | 0.00 | 50.00 |
| Contractor | 53 | 5.8 | 7.2 | 1.0 | 0.10 | 30.00 | 5.46 | 6.87 | 0.94 | 0.00 | 30.00 |
| Project Manager | 37 | 4.3 | 3.9 | 0.6 | 0.50 | 15.00 | 3.64 | 4.37 | 0.71 | 0.00 | 20.00 |
| Total | 161 | 6.4 | 7.8 | 0.6 | 0.10 | 50.00 | 5.62 | 7.18 | 0.56 | 0.00 | 50.00 |

Consultant project managers typically act as the client's representative for projects and thus would possess reasonable knowledge to be familiar with the direct rework costs (in the form of change orders and defects), as they invariably manage time and cost schedules. Project managers, however, may not be aware of the direct rework costs associated with redocumenting aspects of the project after clients have requested design changes and/or omissions. While client involvement in projects has been identified as a factor that can contribute to project success (Walker 1994 ; Chan 1996 ; Love *et al.* 1998b), their involvement may also lead to rework. For example, drawing on the qualitative comments provided by the respondents, a project manager stated that the "client was a decision-maker and actively involved in construction, resulting in scope and design changes throughout the construction". This in turn can lead to design consultants having to re-document or provide additional documentation, which can significantly affect their fee, as they are often not reimbursed for this service (Tilley and McFallan 2000).

Several design consultants articulated this point, with one stating that "reworking of documentation is becoming a common occurrence on projects, which is not reflected in our fee". Another design consultant stated that "a lot of rework had to be done on the documentation to reduce the scope of packages or substitute materials in an attempt to get the project within the budget". Re-documentation due to design changes and omissions initiated by clients and end-users appears to be a regular occurrence in Australian projects (Love *et al.* 2000). Acknowledging this endemic practice, a design consultant stated :

> Rework is an occurrence that consultants try hard to avoid because it can potentially lead to cost increases, which we are often not paid for. So the tendency, therefore, is to hold back on committing resources until all the information required to complete the task is available. Rework, nevertheless, often occurs and can usually be put down to poor planning or devoting of insufficient time to the planning and design before commencing construction.

The allocation of resources and planning of the documentation process are important points that need to be addressed if rework is to be reduced (Love et al, 2000a). Yet understanding why there are differences in the estimates of direct rework costs is a major research task in itself and thus worthy of further investigation. However, it would appear that consultant project managers might not fully understand how changes/omissions could affect the performance of design consultants (particularly the way in which they manage the documentation process) and as such explain the differences in the estimates for direct rework costs.

The descriptive statistics revealed differences between the design consultants (M = 6.77%, SD = 8.34%), contractors (M = 5.46%, SD = 6.87%), and project managers (M = 3.64%, SD = 4.37%) in their estimates of indirect rework costs. Levene's test of homogeneity of variances was not violated ($p < 0.05$), which indicates that the population variances for each

**Table 7. Direct and Indirect Rework Cost for Procurement Methods Used**

| | | Direct rework costs | | | | | Indirect rework costs | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Procurement method | N | Mean | Standard Deviation | Standard Error | Minimum | Maximum | Mean | Standard Deviation | Standard Error | Minimum | Maximum |
| Traditional lump sum | 84 | 7.03 | 8.27 | 0.90 | 0.50 | 50.00 | 5.63 | 6.61 | 0.72 | 0.00 | 30.00 |
| Traditional cost plus | 1 | 20 | ------ | ------ | 20.00 | 20.00 | 10.0 | ------ | 2.76 | 10.00 | 10.00 |
| Traditional with provisional quants | 5 | 10 | 14.28 | 6.38 | 1.00 | 35.00 | 4.50 | 6.18 | 1.00 | 0.00 | 15.00 |
| Design and manage | 2 | 5.5 | 6.36 | 4.50 | 1.00 | 10.00 | 1.0 | 1.14 | 1.11 | 0.00 | 2.00 |
| Construction management | 26 | 4.11 | 4.15 | 0.81 | 0.50 | 15.00 | 4.48 | 5.67 | 1.65 | 0.00 | 20.00 |
| Management contracting | 1 | 2.0 | ------ | ------ | 2.00 | 2.00 | 0.0 | ------ | ------ | 0.00 | 0.00 |
| Design and build | 32 | 5.90 | 7.11 | 1.25 | 0.10 | 30.00 | 6.29 | 9.38 | ------ | 0.00 | 50.00 |
| Novation | 6 | 9.16 | 11.02 | 4.49 | 0.50 | 30.00 | 9.91 | 10.44 | 4.26 | 0.50 | 25.00 |
| Turnkey and package deal | 2 | 4.70 | 0.42 | 0.30 | 4.40 | 5.00 | 8.0 | 9.89 | 7.00 | 1.00 | 15.00 |
| Boot | 2 | 1.5 | 0.70 | 0.50 | 1.00 | 2.00 | 2.0 | 1.41 | 1.00 | 1.00 | 3.00 |
| Total | 161 | 6.44 | 7.78 | 0.61 | 0.10 | 50.00 | 5.62 | 7.18 | 0.56 | 0.00 | 50.00 |

group were equal. The ANOVA test revealed no significant differences between respondents' estimates for indirect rework costs, $F(2,158) = 2.364$, $p < 0.05$. Noteworthy are the design consultants' estimates that are almost twice as much as those estimated by project managers, which again demonstrates the variability associated with rework cost estimates, albeit not for the same projects. For both direct and indirect rework costs, contractors' estimates are approximately midway between those of design consultants and project managers. This implies that contractors may well have a better understanding of actual rework as they invariably operate at the interface between design and construction.

When respondents were asked to compare the rework costs of their selected project with others with which they had been involved, 12% stated that they were comparable "to a very large extent", 16% "to a large extent", 37% "to some extent", 26% "to a minor extent", and 9% "not at all". The mode was found to be "to some extent", and thus it can be concluded that the estimated rework costs reported are generally representative of industry practice in Australia. The rework costs found from this research appear to be relatively comparable to previous studies (Davis et al.1989;, Burati et al. 1992; Abdul-Rahman 1995; Nylen, 1996; Josephson and Hammarlund 1999).However, such studies, with the exception of Barber et al. (2002), fail to differentiate or ac-knowledge the distinction between direct and indirect costs. Thus, the mean rework costs (direct and indirect )reported are considered benchmarks that can be used to pursue best practice in the Australia construction industry. In order to make practitioners aware of the actual costs of rework, a system is needed that can capture and identify rework, such as that developed by Burati *et al.* (1992), Low and Yeo (1998) and Love and Li (2000b), so that appropriate prevention techniques can be identified and implemented in future projects.

### *Rework Costs and Procurement Methods*

Table 7 identifies the direct and indirect rework costs for each procurement method. Levene's test of homogeneity of variances was not violated for direct and indirect costs ($P<0.05$), which indicates that the population variances for each group were equal. The

ANOVA revealed no significant different between procurement methods for direct rework costs, $F(9,151)=1.004$, $p<0.05$ and indirect rework costs $F(9,151)=0.624$, $p<0.05$. No Tukey HSD post hoc tests were performed for both direct and indirect rework costs because at least one procurement method category had fewer than two cases. Consequently, procurement methods were reclassified into traditional and nontraditional. So, to determine whether there was a difference between traditional and nontraditional procurement methods for rework costs (direct and indirect), a t-test was undertaken. Table 8 present the mean and standard deviation for total rework costs for traditional and nontraditional methods, and result of the t-test are presented in Table 9. At the 95% confidence level, no significant difference in the total cost of rework was experienced in project using different procurement methods. Therefore, it is concluded that rework costs do not significantly vary among procurement methods employed.

### *Rework Costs and Project Types*

Refurbishment and renovation projects are considered prone to higher rework costs than new build projects because of the degree of uncertainty and complexity associated with the building work to be undertaken (Love and Wyatt 1997). In fact, NEDO (1988) and Naoum and Mustapha (1994) indicate that facility types are linked to the concept of complexity and thus have influence on project performance. The direct and indirect rework costs for the various project types sampled in this research can be seen in Table 10.

**Table 8. Total Rework Costs for Traditional and Nontraditional Methods**

| Procurement method | N | Mean | Standard deviation | Standard Error mean |
|---|---|---|---|---|
| Traditional | 89 | 12.77 | 13.50 | 1.43 |
| Nontraditional | 72 | 11.10 | 13.66 | 1.61 |

**Table 9. t-Test for Procurement Methods and total Rework Costs**

| | Levene's Test for Equality of variances | | t-Test for equality of means | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | F | Sig. | t | df | Sig. (2-tailed) | Mean difference | Standard Error difference | 95% CI of difference | |
| | | | | | | | | Lower | Upper |
| Equal variances assumed | 0.136 | 0.713 | 0.772 | 159.000 | 0.441 | 1.662 | 2.152 | -2.589 | 5.912 |
| Equal variances not assumed | | | 0.771 | 151.285 | 0.442 | 1.662 | 2.155 | -2.596 | 5.919 |

**Table 10. Direct and Indirect Rework Cost for Project Types**

| | | Direct rework costs | | | | | Indirect rework costs | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Project Type | N | Mean | Standard Deviation | Standard Error | Minimum | Maximum | Mean | Standard Deviation | Standard Error | Minimum | Maximum |
| New build | 90 | 6.10 | 7.18 | 0.75 | 0.10 | 35.00 | 5.69 | 7.70 | 0.81 | 0.00 | 50.00 |
| Refurbishment/Renovation | 43 | 7.29 | 9.73 | 1.48 | 0.50 | 50.00 | 5.60 | 6.43 | 0.98 | 0.00 | 30.00 |
| Fit-out | 14 | 7.78 | 7.70 | 2.06 | 1.00 | 30.00 | 6.10 | 7.90 | 2.11 | 0.50 | 30.00 |
| New build/refurbishment | 11 | 4.95 | 4.67 | 1.41 | 0.50 | 15.00 | 5.81 | 5.92 | 1.78 | 0.00 | 20.00 |
| Combination of all | 3 | 3.33 | 1.52 | 0.88 | 2.00 | 5.00 | 0.66 | 0.577 | 0.33 | 0.00 | 1.00 |
| Total | 161 | 6.44 | 7.78 | 0.61 | 0.10 | 50.00 | 5.62 | 7.18 | 0.56 | 0.00 | 50.00 |

The ANOVA test was used to determine if there was a significant difference between project type and rework costs. Levene's test of homogeneity of variances for both direct and indirect rework cost was not violated ($p < 0.05$), which indicates that the population variances

for each project type were equal. The ANOVA test revealed no significant differences between project types for direct rework costs $F(4,156) = 0.489$, $p < 0.05$, indirect rework costs $F(4,156) = 0.371$, $p < 0.05$, and total rework costs, $F(4,156) = 0.824$, $p < 0.05$. Considering the results of the ANOVA test, it is concluded that rework costs do not significantly vary among project types.

**Conclusions**

The research presented in this paper set out to determine if there were significant differences in rework costs for different project types and procurement methods. Data from 161 Australian construction projects were obtained through a questionnaire survey. It was revealed that traditional lump sum methods are the most popular forms of procurement used in Australia, despite calls for the adoption of more integrated methods, such as design and build. Considering this finding, cost certainty – an inherent feature of the traditional lump sum method – appears to be a key driver for clients. This does not, however preclude clients from demanding earlier completion as well. Similarly, pressures to be reduce the time to design and produce contract documentation under this method can be compressed to meet client demands. These client demands may influence the quality of contract documentation produced, as errors and omissions may emerge that can result in rework and thus cause cost and schedule overruns.

The cost and schedule growths for the projects sampled were calculated, and it was found that mean cost and schedule growths were 12.6% and 20.7%, respectively. While some projects recorded relatively high rework costs, others also experienced negative cost and schedule growths, which was an interesting and unexpected finding. This implies that the original contract period may have contained "buffers", or items of work were deleted, or construction methods were altered (or a combination of all) so that the project could be delivered within budget and on time (or less).

The analysis revealed no significant differences between the cost and schedule growths on the projects sampled. In addition, cost and schedule growths were found to be significantly correlated with direct rework costs, which suggests that rework can adversely influence project performance. In examining this further, rework as a percentage of cost growth was calculated and found to amount to 52.1% of the total, so it can be concluded that rework can make a significant contribution to a project's cost overrun. Mean direct and indirect rework costs were found to be 6.4% and 5.6% of the original contract value, respectively. Nevertheless, rework costs were found not to vary significantly with project type and procurement method used. Yet there is a great need to reduce rework costs if projects are to improve their productivity and performance. In particular, the influences of an organization's management practices (e. g. quality and learning) and project management strategies on reducing rework costs should be examined. While this research has provided an insight into practitioners' estimates of rework costs, the "actual" costs still remain relatively unexplored in construction.

*(Courtesy : "American Society of Civil Engineers". Journal of Construction of Engineering and Management / January/February 2002)*

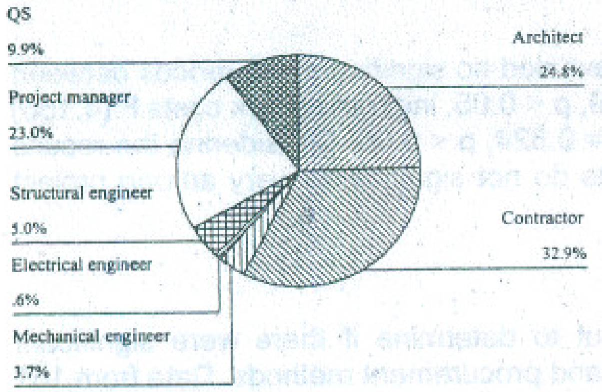



**Fig. 1.** Respondents by profession

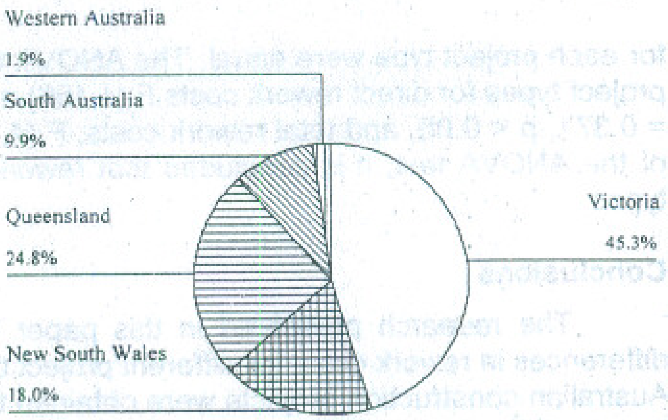



**Fig. 2.** Respondents by state

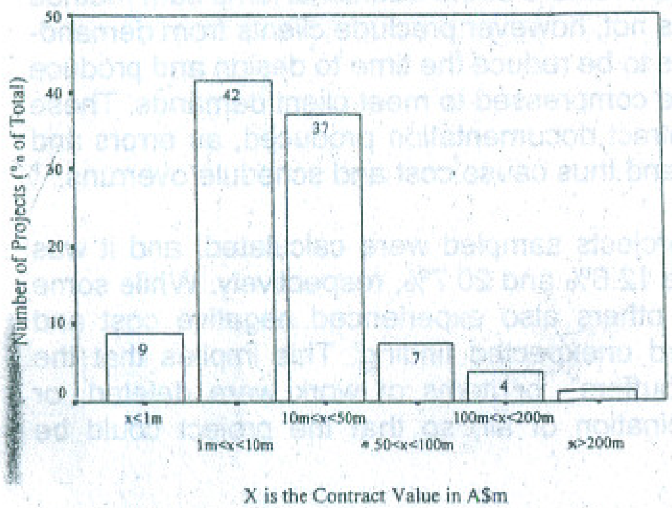



**Fig. 3.** Number of projects by contract value

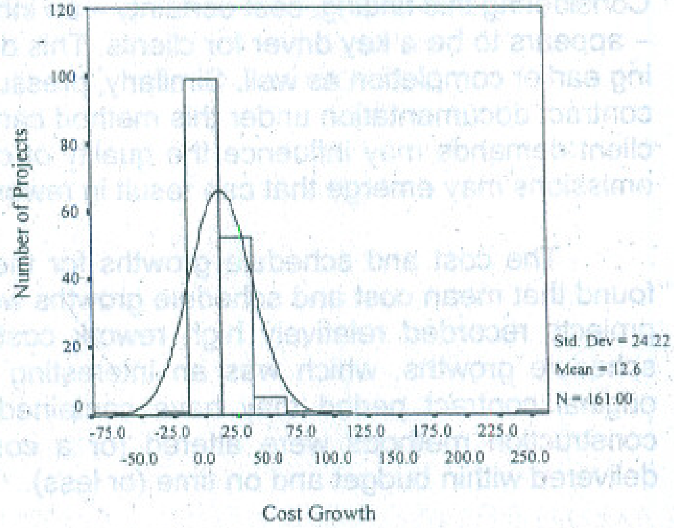



**Fig. 4.** Percentage cost growth for projects

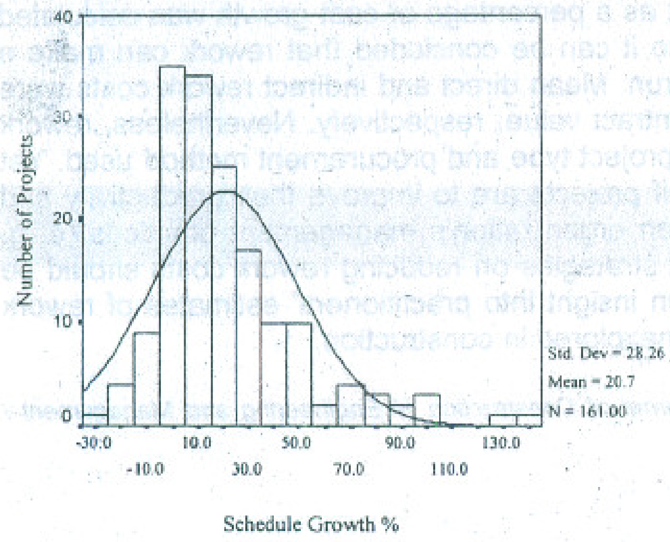



**Fig. 5.** Percentage schedule growth for projects

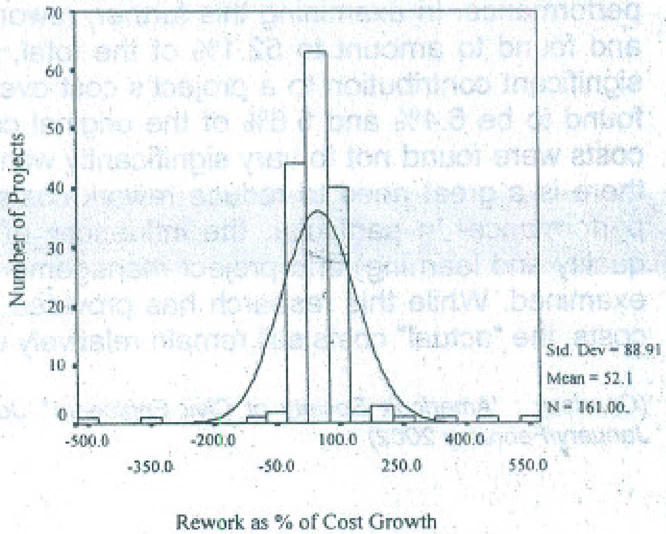



**Fig. 6.** Rework as percentage of cost growth

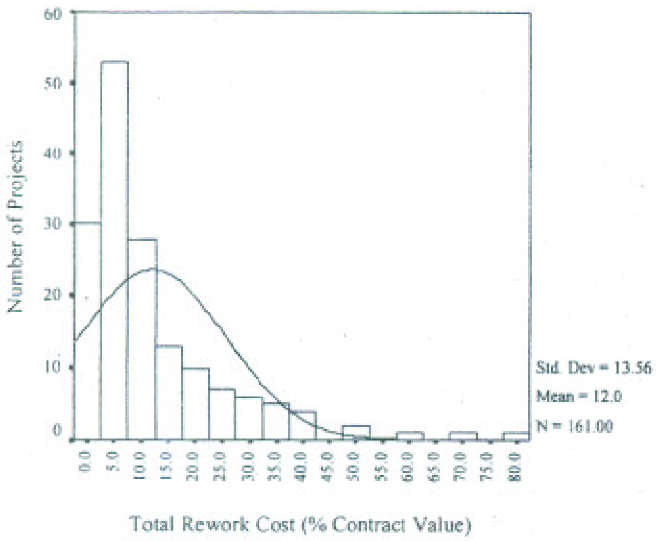



**Fig. 7.** Total rework costs as percentage of original contract value

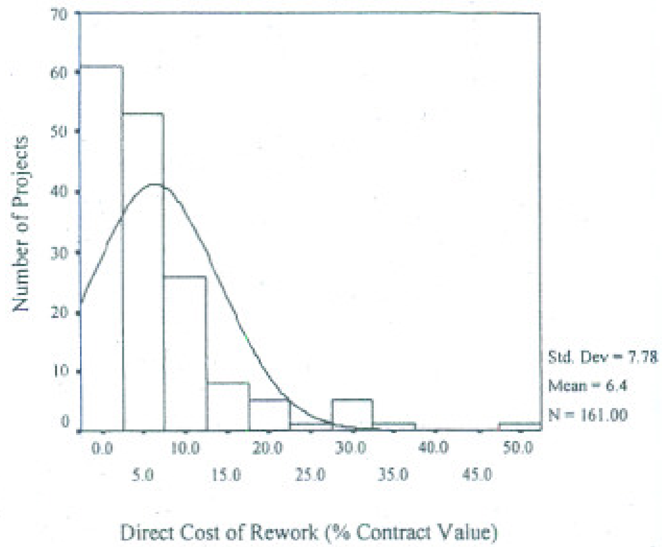



**Fig. 8.** Direct rework costs as percentage of original contract value

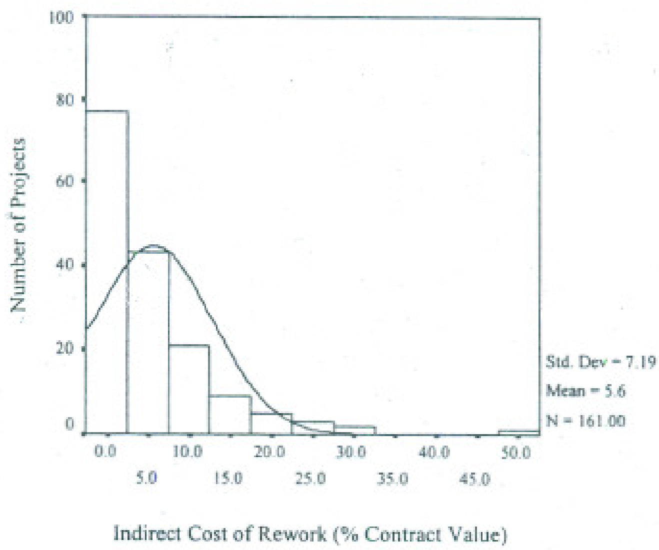



**Fig. 9.** Indirect rework costs as percentage of original contract value

# GROUNDWATER : POTENTIAL AND CONSTRAINTS

By

**MARCUS MOENCH**

Groundwater problems emerging in many parts of the world reduce drought-buffer supplies, threaten environmental values, and increase risks for many of the world's poorest people. Programs to improve public understanding and basic scientific information regarding the resource base and to encourage the evolution of groundwater management systems are essential. Furthermore, because many countries will need years to develop systems for managing groundwater, policies should encourage users to adapt to water scarcity conditions rather than attempt to solve water problems per se.

## NATURE AND SCOPE OF GROUNDWATER PROBLEMS

The Green Revolution, which led to dramatic increases in food production, especially in Asia, has also been called the "tube-well" revolution" because part of what drove it was groundwater development on an unprecedented scale. The number of mechanized wells in India grew from a few thousand at the time of independence in 1947 to tens of millions today. As a buffer against drought and precipitation variability, groundwater plays a critical role in food and livelihood security. Increased access to groundwater has reduced risks substantially, enabling many farmers to move out of poverty. All such benefits, however, may have come at a high cost.

In recent summers, flows from the world's second-largest spring, the headwaters of the Al-Khabour River in Syria, have declined from a long-term average of 50 cubic meters per second to only a few cubic meters per second. Similar stories are emerging in many regions. Groundwater-level declines of 1 – 3 meters per year are commonly reported for monitored wells in arid and semi-arid regions. In extreme situations, such as Sana'a in Yemen, shallow aquifers are almost depleted. Efforts to tap new groundwater supplies in Sana'a have been unsuccessful despite the drilling of wells that exceed two kilometers in depth. In the Middle East and North Africa, it is no longer uncommon to encounter wells drilled to depths that have historically only been seen in the oil industry. Even in humid regions such as Bangladesh, water-level fluctuations may be increasing, and water is scarce during the dry season. Furthermore, scarcity and quality concerns are linked. Groundwater pumping often mobilizes water that is saline or contains natural contaminants such as arsenic or fluoride. When combined with increasing pollutant loads from agriculture, industry, and municipal sewage, this pumping can irreversibly contaminate aquifers.

The threats to groundwater resources are clear, but their extent is far less so. Worldwide, most groundwater monitoring networks are relatively new and collect a limited array of data on water levels and basic water quality parameters. Detailed data on water quality and pollutants are rarely available except in relation to specific local concerns. Data are also often unavailable on critical components of the water balance – such as groundwater extraction, evapotranspiration from native vegetation, and deep inflows to aquifers. Furthermore, data on water-level changes can be misleading. Aquifers can take tens to hundreds of

years to equilibrate when disturbed. As a result, short-term water-level declines do not necessarily indicate overdraft.

Nonetheless, water-level changes and fluctuations are the most important factors influencing access to groundwater for the environment and for human uses. For small farmers along the Ganges, the deep saturated basin beneath their fields is irrelevant. What they care about is whether or not water is available within the tens-of-meters range from which they can afford to pump. When levels decline below or fluctuate outside that range, farmers lose access to irrigation and households may lose access to drinking water. Fluctuations are equally important from an environmental perspective. Stream flows and wetlands often depend on high groundwater levels. Even modest seasonal declines can affect surface water bodies severely. Water quality is also affected when changing levels mobilize low-quality water or cause waterlogging and associated salinization problems. From the perspective of current usage, this dynamic – the interaction between groundwater levels, underground flow patterns, surface water bodies, and the economics of groundwater access – is far more than the overall balance between extraction and recharge within an aquifer. Moreover, this dynamic is highly variable, heavily dependent on local conditions, and often missed in the data sets collected by groundwater departments.

## OPTIONS FOR MANAGING GROUNDWATER

The lack of information and understanding regarding groundwater dynamics presents a major challenge for those developing effective management systems. The challenge is as much social as technical. Without both data and a shared understanding of the problems, the social consensus needed to implement decisions is difficult to generate – and groundwater management decisions are often difficult.

Standard approaches to reducing groundwater overdraft, for example, often require metering of all wells, establishment of formal water rights, and regulatory and economic mechanisms to bring extraction down to sustainable levels. While progress toward this goal has been made in a few water-scarce countries such as Jordan and Israel, the situation is more complex in locations (such as India) with tens of millions of wells and conditions that vary greatly even at local levels. Even where managements is most advanced it is socially and politically difficult to reduce groundwater extraction to sustainable levels. Wells are generally private and highly dispersed. Inventorying them and monitoring extraction are problematic. Furthermore, reducing use to sustainable levels in arid regions often requires substantial reductions in extraction, which can have tremendous economic and social impact. As a result, governments are not inclined to force reductions.

Groundwater experts often propose community management of groundwater as an alternative to state regulation. Although globál experience in this area is limited, experience with resources other than water indicates that several factors are critical to the success of community management – for example, clear boundaries on the resources and its user group, the ability to control free riders, and information on the use and condition of the resource. Such factors are difficult to establish in the case of groundwater.

Successful groundwater management has been achieved through intermediate-level institutions such as the quasi-government groundwater districts in the Western United States and somewhat similar organizations in parts of France. Organizations of this type hold much promise. Their development, however, often takes decades, and to be effective they require data, technical capacity, and some degree of supporting social consensus.

Markets are also central to any framework for groundwater management. Water markets in the Western United States are based on water rights systems that attempt to quantify the volume sustainably available in aquifers and allocate it among users. Transactions involve contracts and often the formal transfer of the water right as well as the water itself. In developing countries formal water rights are rarely involved. Instead, informal transactions occur between well owners and adjacent farmers for irrigation or tanker companies that deliver the water to urban customers. Prices for irrigation are low compared with tanker deliveries. In 1986 in Yemen, for example, the cost of extraction near Ta'iz was approximately US $ 0.0005 per cubic meter. Rural irrigators paid US $ 0.02-0.05 per cubic meter ; urban bulk users paid an average of US $ 2.60 per cubic meter for tanker supplies ; and smaller customers paid as much as $ 25.00 cubic meter for purified groundwater. That pattern is typical, the urban poor generally pay the highest prices.

Two points merit attention, First, because formal rights are not involved, prices reflect short-term pumping capacity, not the longer-term sustainability of extraction rates. Second, even with rights systems, markets indicate the value individual users gain from extracting groundwater but not the economic, environmental, and sustainable use values that accrue when groundwater is left in place.

Aside from groundwater market prices, energy is the primary variable cost affecting groundwater extraction. In countries such as India, energy subsidies were used to encourage groundwater development during the Green Revolution. Now, despite water-level declines and huge effects on state budgets, those subsidies have proved politically difficult to eliminate. As a result, agriculture now officially consumes more than 50% of total power production in some Indian states. Appropriately structured energy prices can provide users with a major incentive to use groundwater more efficiently. Still, given the high yields associated with groundwater and the numerous factors affecting the economics of agriculture, energy prices alone cannot be expected to reduce groundwater extraction to sustainable levels.

**ACTION IN THE SHORT AND LONG TERM**

How can emerging threats to the groundwater resource base be addressed ? Most groundwater experts advocate the development integrated management systems. While important, such efforts require long-term data on aquifer conditions along with well-established institutional capacities that are unavailable in many regions. Therefore, integrated management initiatives rarely generate results over the short term. Alternative approaches in particular those that encourage populations to adapt to conditions of water scarcity and to reduce pressure on the resource base – are essential. Existing coping strategies such as the migration of populations out of agriculture and into urban areas along with the development of non-agricutural economic systems – represent a starting point for reducing pressure on areas where overdraft levels are high. Although they do not ensure sustainability of the resource base, such strategies can provide essential breathing space for the longer-term development of management institutions.

*Courtesy : Institutional Food Policy Research Institute USA Focus-9 Overcoming Water Scarcity and Quality Constraints*

# AN APOLOGY FOR LIME

By

**Shaukat Ali**
General Manager (Civil)
Nazir & Company (Private) Limited Lahore

## Synopsis

Lime as the main cementing constituents of masonry mortars is one of the oldest of construction materials, having been used more than 2500 years before Christ in unearthed construction of Mohenjodaro and Harappa. Lime was extensively used as a highly cost effective cementing material for masonry and plastering work even before partition of Indo-Pakistan sub-continent.

With the advent of Portland Cement and its subsequent wanton and blatant use lime was thrown into oblivion by the Engineers and its use became confined to white or colour washing alone, as it could not match with the Portland Cement in setting as well as in early development of strength.

In Pakistan there is a great variety of sands. Barring a few sources sands are generally very fine and not fit for even cement mortars what to speak of concrete. For fine sands being used in cement mortars, lime is a sine qua non and a vital admixture, if the mal-effects of sand-flow and poor mortar strength are to be precluded. In this paper the dire necessity of use of lime in cement sand mortars of very fine sands has been emphasized. An attempt has also been made to highlight the urgency and exigency of use of lime for soil stabilization of clayey soils in road construction to minimize the haulage of stone aggregate from long distances. Engineers and Architects have also been reminded how lime can be used as a very cost effective cementing additive with ground burnt clay (surkhi), fly-ash, rice husk ash (pulverized), blast furnace slag and other pozzolanic materials naturally available or available as industrial waste for low cost housing and other such constructions where cost has to be minimized.

## Historical Background

Sometimes around 8000 B. C. somewhere in Middle East, probably in Mesopotamia, man began to cluster in tiny settlements. Gradually these settlements grew into hamlets, hamlets into villages and villages into cities. By this time man had known the use of mud, wood, stone and bricks to make his dwellings. Brick sewers found in Nineveh indicate that man had become conversant with the use of burnt bricks around 4000 B. C. The technique of brick masonry flourished with the Egyptians, Babylonians, Indians, Chinese, Greeks and Romans and this art reached its climax especially with those civilizations which had no stone.

The forerunners of the brick-layers of today knew the philosophy behind the use of mortars and in the beginning laid their bricks in mud as it provided a uniform bedding and kept the bricks in the desired position besides filling the joints and making the masonry safe against the ingress of rain water or the inclement cold winter winds.

With the passage of time, man found mud mortar still useful but began to think of some superior mortars which could match the bricks in strength and could be more resistant to weather. The Babylonians were the first to use asphalt as mortar as replacement to mud. This was definitely a superior mortar and had no equal in a number of ways ; but its use was restricted to a relatively small ambit because asphalt was not available everywhere and its natural deposits were limited.

During the process of burning of bricks, man came across lime, first rejected it as a useless trash but latter his knowledge about it increased and he began to use it with sand, burnt clay, volcanic ash, cinders and pozzolanos as brick masonry mortar in preference to mud. As time passed, lime mortars became very popular and began to be used most widely. In Indo-Pakistan subcontinent, lime has been used in mortars 4500 years ago and remnants of structures found in Mohenjodaro, dating back to 2500 B. C., speak for themselves that lime mortars could stand the rigours of time. Romans were the first to observe cementing properties of lime with finely ground pozzolanos or burnt clay and extensively made use of this property in building structures like aquaducts, collossea and amphitheaters.

In most parts of the world ordinary and hydralic lime remained in vogue millennium after millennium and century after century until an English brick layer, James Aspdin in 1824 invented modern day cement by burning chalk with clay and grinding the resultant clinker into a fine powder. More or less around this time in history, engineers in various parts of the world especially Europe and the U. S. A. had discovered cement as dependable bonding ingredient for mortars.

Cement had apparent supremacy over lime as it could set in a few hours, start hardening and acquiring strength which lime took years to do so. Engineers were greatly impressed by cement and wherever cement could be procured easily it began to replace lime. Ultimately a time came when lime fell into oblivion. At the moment lime has almost been forgotten and is only used for white or colour washing.

**Use of Lime in Mortars**

There is, however, urgent need for use of lime in mortars even in cement mortars and this need is more exigent than it ever was especially in those areas of Pakistan where good quality sand is not available. This is all the more necessary because most of the sands available in Pakistan excepting Malir and Bholari sand in Sind, Haro and Lawerencepur sand in Punjab and Khor or Nullah sands in N. W. F. P., are either too fine or too coarse to meet the structural requirements of mortars and concretes. Some of them almost touch the border of silt like the Ravi sand and having a fineness modulus never exceeding 1. From the structural point of view Ravi sand is not suitable even for cement mortars what to speak of its use in cement concretes. As per state-of-the-art latest specifications of sands for cement concretes, the minimum fineness modulus of sand is around 2.30 while the maximum never exceeds 4.0. For mortars the fineness modulus requirements are generally not specified because the minimum structural strength of masonry mortars is never asked for. Now the fineness modulus range of 2.30 to 4.0 gives an indication of quality sand from the point of view of coarseness for concretes but this range would not be suitable for mortars, masonry work and not at all for plaster work. The most appropriate fineness modulus for masonry mortars is around 1.5 while for plaster work it can go upto 1.20 or even 1.0, though finer the modulus lesser would be the mortar strength for the same cement sand ratio. In a situation like ours where most of the sands are fine and coarse sands have to be hauled from long distances, lime is the only cheap cement sand mortar additive which can neutralize the mal-effects of its fineness and can impart desirable qualities in the mortar in most of the cases with substantial reduction in cost. The efficacy of lime as a cement sand mortar when using fine sands would be evident when we examine the issue at greater depth.

**Textural characteristics of sand in cement-sand Mortars**

In cement sand mortars, cement in the form of a paste, performs two basic functions of covering the surface area of sand and filling its interstices and voids. The strength of mortars depends upon the extent the surface of the individual particles is covered and the voids between them are filled. For a given sand or fine aggregate, there is certain definite amount of cement which can cover the surface area of the particles and fill their inter-granular space. This amount of cement per unit volume of the sand can be termed as the SCR or Spesific-Cement-Requirment of that sand. If other parameters for mortar strength remain unchanged, only the quantity of cement equivalent to SCR can give the maximum strength. If the amount of cement is brought down from the SCR level, the process of surface encompassing and void filling will decrease and the strength will be reduced. If it is further reduced, the strength will come down further and at a certain stage the cement paste will only partially cover the surface area without filling the voids. Such a mortar or concrete is apt to display sand-flow when its skin is disturbed by scratching with a coin or key betraying its very poor quality. If, however, the amount of cement is increased beyond the SCR stage, the cement paste shall have more volume than the intergranular space which exists in the sand when it is closely packed. This increased space shall be occupied by the cement paste. In rich cement-sand mortars, cement paste shrinks on drying giving rise to shrinkage cracks. In poorer cement sand mortars, cement paste occupies less space than the space available between the grains and the shrinkage effect remains localized and does not appear on the surface.

Of the three constituents of mortars, namely cement, sand and water, cement is the most expensive and if on the basis of more intimate knowledge of the surface area and voids, the quantity of cement is varied according to the actual SCR of a specific sand, the approach would be more scientific, economical and satisfying. The present practice of adhering to the same arbitrary cement sand ratios for different sands at different localities is technically not a sound engineering practice.

That the sands of different fineness moduli require different quantities of cement can be very easily shown, if we study the limits within which the usable sands are found in nature. As per British, French, German, Swiss and Massachusettes Institute of Technology systems of classification, a soil can be classified as sand or gravel when its particles lie within the following broad limits :

| | | |
|---|---|---|
| Gravel | -- | Diameter greater than 2.0 mm |
| Coarse Sand | -- | Diameter between 2.0 mm to 0.6 mm |
| Medium Sand | -- | Diameter between 0.6 mm to 0.2 mm |
| Fine Sand | -- | Diameter between 0.2 mm to 0.06 mm |

According to the above, the grain size of the coarsest sand is 33.33 times that of the finest. If for the sake of argument the sand grains are assumed to be cubical shape and the 2.0 mm cubes of coarse sand are successively sub-divided into cubes of 0.06 mm, the surface area will increase from 24 sq. mm to 800 sq. mm. Obviously if X grams of cement are required to cover surface area of 24 sq. mm of sand, 33.33 X grams of cement will be needed to cover the surface area of 800 sq. mm. Very fine sands therefore may require about 30 times more cement than very coarse ones.

The above statement is only valid if all the grains of the coarsest and finest sand are of the same size. Since sands occurring in nature are usually a mélange of coarse and fine particles, the actual difference of surface area and concequently of cement is always far less than the above.

A predominantly coarse sand, however, having certain percentage of medium and fine sand will require less cement than a predominantly medium or fine sand having sand particles graded within its range. A well graded coarse sand will require less cement than a well graded medium or fine sand because the surface area of the coarse sand will be less than that of the medium or fine sand. When a sand is well graded, the interstices between the bigger particles are filled by smaller particles and so the amount of voids to be filled by cement are reduced but the surface area of a coarse well graded sand still remains less than that of a medium or fine well graded sand.

The quantity of voids present in a given sand depends upon the packing of sand grains, their shapes and surface area. In the loosest state ordinary sand can have void ratio as high as 0.85 while in the densest state 0.40. (The densest state is only a theoretical state which can be approached in the labbratory but which can hardly be achieved in the field due to practical handicaps). As the sands particles become finer and finer, the void ratio also increases. This would become apparent if we again consider the successive sub-division of a 2.0 mm cube coarse sand into very small grains, say 0.001 mm or 1 micron cubes. In this case the surface area increases from 24 sq. mm to 48000 sq. mm and the sand ceases to behave as a conglomeration of cohesionless particles but will act as a plastic clay having certain amount of cohesion. Since clays or silts possess high void ratio as compared to sands, it goes without saying that the void ratio of sand will increase with reduction in grain size or increase in the surface area. At this juncture it will also be worthwhile to highlight the parameter of fineness modulus which is a rough and crude indicator of coarseness of sand and fine aggregates. Fineness modulus is determined in the laboratory through simple sieve analysis. Its high value is generally indicative of a coarser sand having lesser surface area and a low value envisages a fine sand of large surface area. Fineness modulus of fine aggregates and sands is inversely proportional to surface area and directly to particle size.

In the context of sands it will not be out of place to give recommendations of the various codes of practice. The German code of Practice DIN 1045 makes definite recommendations about the type of sand gradings which are very good or suitable for concrete work. The grading of sand is very good when the fineness modulus lies between 3.50 to 4.40 and suitable when the modulus is between 2.60 to 3.50. The American Society for Testing Materials considers the sands lying within a fineness modulus of 2.30 to 3.38 as suitable. Similarly the British Code of Pratice classifies sands suitable for concrete and masonry work in four different zones. Sands

falling in any of the four zones are considered as suitable except that of zone 4 which is recommended to be tested before use.

In Pakistan the major sources of sands with their approximate fineness moduli are as under :

| Source | Fineness Modulus |
|---|---|
| Haro and Lawerencepur sand | 1.91 to 3.50 |
| Sohan River sand | 0.88 to 1.06 |
| Chenab River sand | 1.00 to 1.50 |
| Ravi River sand | 0.80 to 1.00 |
| Malir River sand | 3.50 to 4.60 |
| Bholari sand | 3.20 to 3.95 |
| Sui Quarry | 2.70 to 3.10 |
| Sakhi Sarwar and D. G. Khan | 1.50 to 2.50 |
| D. I. Khan (Indus River) | 0.64 to 0.80 |
| Isa Khel (Kalabagh) | 0.80 to 0.89 |

The above inventory of sources is by no mean comprehensive but, in general, sands available in Pakistan are too fine to be used in quality concrete or in quality mortars.

The use of low fineness modulus sands having high cement requirements coupled with hand mixing gives rise to a cement sand mortar in which the surface of the sand grains is not completely covered by the binding cement paste. On setting and hardening, the sand at places begins to flow or travel when the skin of such mortar is disturbed by scratching with a coin or key. In low cement sand mortars such as a phenomenon is apt to occur even though the mortar is machine mixed. From the structural point of view, the possibility of sand-flow is highly undesirable as it betrays a mortar of too poor a compressive strength and cement sand ratio to give sound construction.

Due to high cost of transportation of suitable sands and vast sources of easily and cheaply available unsuitable fine sands, it is essential to bring down the already high cost of construction and to find out ways and means to make these fine sands acceptable and usable at least for cement sand mortars. There can be so many ways to do this. One is to increase the quantity of cement till the minimum acceptable mortar strength is reached. The other method can be of increasing the covering capacity of cement itself by grinding it further or by adding a suitable admixture.

The surface area of ordinary Portland cement varies from 2250 to 2500 sq. cm/gm and its grain size ranges from 7.5 microns to 60 microns. Theoretically it is quite possible to double the surface area of the cement but whether it is economically feasible to manufacture cement of such fineness with the existing manufacturing set up is very difficult to say. In view of this there is no other alternative but to think of a suitable admixture which should be preferably indigenous and inexpensive. Both technically and economically lime can be very useful admixture as it can neutralize the detriment resulting from fineness at least in cement mortars both for masonry and plastering work.

### Covering characteristics of Lime/Workability

When 100.09 lbs of limestone, say marble, is heated at 1000 F, 44.01 lbs of carbon dioxide gas is produced leaving 56.08 lbs of lime or calcium oxide (assuming of course that limestone contains no impurities and is 100% calcium carbonate). This lime, in its un-slaked state, has great avidity for water because it is chemically highly unstable unless it gets hydrated into calcium hydroxide. For complete hydration, it requires only 32.10% of its weight of water but in order to make a workable putty for mortars it may need 250% to 300% of its weight of water and its volume may increase to 2.5 time its original volume. This lime putty when pure is completely miscible with cement and is soluble in water. It is this characteristic of lime coupled with its quality to harden on carbonation, which gives it a tremendous covering capacity which cement because of its limited fineness can never achieve. With 10% to 20% of lime putty by weight of cement the optimum quantity of which for a given fine sand can be determined in the laboratory, the sand-flow can be effectively stopped and other desirable qualities can be imparted. Specifically in case of low cost housing very low cement sand ratios can be employed by further decreasing the quantity of cement and slightly increasing the quantity of lime putty or lime hydrate.

In putty form, every molecule of lime is surrounded by a number of molecules of water as the quantity of water in the putty, in excess of that required for complete hydration is usually high (250% to 300%). The molecules of water act as perfect lubricants for the free movement of lime molecules. This free movement or the ease of one particle to slide over the other gives rise to a state of workability and plasticity in the in the lime mortar which would require many-fold increase of cement alone to acquire. With the addition of lime as an admixture, the mortar can be laid and spread more easily. This helps to reduce the labour of brick-laying. Especially in case of plastering, lime is indispensable if economy is to be affected and sand-flow is to be checked. It is a plasterer's pleasure to plaster with cement mortars containing lime as an admixture because it gives him great ease in applying the trowel or float and getting the desired results of evenness and flawlessness in the resultant surface. Besides, because of its greater adhesiveness and plasticity, there are less droppings and less wastage and even the droppings can be re-used because of the slowing of mortar hardening.

### Merits of Lime Mortars

During the operation of laying bricks a mortar containing lime, because of its high workability and plasticity, does not span the voids or gaps between the perpends but it slumps into them giving a relatively better bond between the bricks and the mortar. From both practical and structural point of view this characteristic of lime mortar is of great significance, because a contractor will find it harder to deceive the field staff by lipping the perpends and joints only on the face and leaving the space behind without mortar as the mortar in the perpends will tend to slump down unless it is completely filled.

Another very important facet of lime is that it has great capacity for retaining water. This property in mortars is known as water retentivity. Water retentivity of a mortar is the property to retain water in excess of that required for hydration. This excess water plays a very important role between the mortar and the absorbent bricks, the mortar having water retentivity stiffens quickly as the water is absorbed by the bricks. In this way the mortar goes to the brick pores and voids resulting into a greater bond between mortar and the bricks. Highly water-retentive mortar is somewhat self-curing in the initial stages and is thus helpful in this way also.

In mortars there is practically no shrinkage due to sand ; it is either due to cement or lime. Lime shrinks 3 times more than cement. It can be argued that addition of lime to cement sand mortars will increase shrinkage and may lead to microscopic cracks. The cracks will appear if there is enough of binding paste between every two grains of sand, but if the binding paste is not enough because of the low fineness modulus and high cement requirements of the sand, the grains of sand will be just in contact with each other and the space between them will be hardly filled ; with the result that no shrinkage would occur. If however, some shrinkage is noticed, then quantity of lime can be reduced to control it.

As a result of my firm conviction in lime as a panacea of ills of fine sands, I constructed my own residence using Ravi sand and waste lime of Pakistan Oxygen Limited for all masonry and plaster work with a little sizing of cement to accelerate its slow rate of hardening. For masonry mortars the ratio was one part slacked lime in liquid form, ten parts of Ravi sand and quarter part cement with adequate lime water to make a workable mortar while in case of plasters the quantity of cement was doubled to that of the masonry mortars.

This construction was highly cost-effective as the lime was free and only needed to be hauled from the factory near Engineering University G. T. Road, Lahore. This type of construction which I could experiment with my own residence proved to be much sounder than that of normal cement sand mortars as part of it simply refused to get demolished after two decades as if it was not masonry work but concrete and this too while dismantling the garage where I had used still poorer mortar than stated above since it was built at the end with left over lime and sand and very little cement.

Of all the types of rocks and stone aggregates found in Pakistan limestone is the most abundant throughout the length and breadth of the country and as such lime as a construction material would always be cost-effective and cheap and there is immediate need to revive its use for the sake of sound construction and economizing in cost.

**Lime as a Constituent of Cement**

It should not be surprising to know that no cement can be manufactured without lime and that in all cements being manufactured two third of their weight is calcium oxide or lime. In other words in each bag of cement we buy no less than 33 Kg lime, and this lime is its integral and inseparable part. Two millennia from to-day the Romans have been using lime for all their masonry work in aquaducts, amphitheaters and colossea and they knew the cementing qualities of lime with finely ground pozzolanos. It is now an established scientific fact that lime when wet, mixes up with finely ground argillaceous and siliceous materials and makes complex calcium alumino-silicate compounds which have cementing properties. Most of the pozzolanos materials contain various percentages of aluminium and silicon oxides and readily mix with wet lime to make compounds which have cementing properties like cement. Use of lime with surkhi (ground bricks) and cinder when properly ground, produced the same cementing effect what the Romans got with pozzolanos. Now-a-days the most economic, cost-effective and endemic pozzolanic material is rice-husk ash which is almost 90% silicon dioxide and can effectively be used with lime to make cheap mortars for brick masonry and plastering. Rice-husk ash can also be used in cement concrete as a cheap additive to utilize the free lime which is always present to some extent in all cement concrete mixes. Rice-husk ash would also neutralize the possible alkali reaction. But its crystalline structure needs to be pulverized before use in all the above cases.

**Use of Lime as stabilizing agent in roads**

Yet another use of lime is in the modification and stabilization of soils for road construction especially when clayey soils are encountered. Soil stabilization with lime would be the only means of cheap road construction in the next decades when haulage of stone aggregates would become too expensive because of the long distances involved and increase in cost of P. O. L. when the nearer sources of aggregates would get depleted as it has already happended with the quartzite quarry of Shahkot.

Lime hardens as it absorbs carbon dioxide from the air when wet but this hardening process is fairly slow and the strength developed as a result of hardening from carbonation is structurally too dilatory to be adequate. This hardening process however gets quite improved in case of reaction of lime hydrate with pozzolanic materials like rice husk-ash or fine ground blast furnace slag from Karachi Steel Mils. The Romans / Italians were the first to know this lime-pozzolanic cementation effect and made use of it both within and outside Italy as major component of road crust. Pavital mixture used on G.T. Road from Shahdara to Muridke as overlaying crust on the existing road is practical use of pozzolanic materials with lime without any secret material.

**Scope of further Research**

Lime is not only a sine qua non for fine sands, it will also become an essential and integral part of lime-ash mortar for low-cost housing in the next millennium when engineers would get convinced with the cementation properties of calcium-alumino-silicates which get easily formed when using limes with silica ash, fly-ash, rice-husk ash or even cinder ash whichever is cheaply available.

Even burned clay when ground and mixed with lime would evince cementing properties. Natural hydraulic lime can be produced if limestone and clay in a mixed state are burned and ground even in indigenous grindstones or chakkies.

$$\text{Limestone + Clay} \xrightarrow[\text{500}^\circ\text{ C and grind}]{\text{heat at above}} \text{Hydraulic Lime}$$

Building Research Station Lahore, Central Testing Laboratory WAPDA and other laboratories in the country can easily carry out research and advise both public and private sectors the benefits of using lime, which has almost fallen into oblivion so far as it primary use for masonry mortars and plastering work is concerned though its secondary role of white and colour washing still continues.

## INSTRUCTION FOR CONTRIBUTORS TO NEWS JOURNAL

The editors of the News Journal of Pakistan Engineering Congress welcome research papers as well as engineering notes and other writings pertinent to the field of engineering and allied disciplines. Your adherence to the instructions printed here will help to ensure that your manuscript receives the attention that it deserves.

**Manuscript Submission**

Please submit one hard copy and one electronic version (on 3.5" diskette) of your manuscript to the Chief Editor, News Journal, Pakistan Engineering Congress, Lahore. The manuscript should include complete address(es) of the author(s), telephone and fax numbers (if any) and e-mail address(es) where available. If there are more than one authors, indicate in the covering letter who is the contact author.

Contributed manuscripts should be typed double spaced, on one side of A4 size white paper, with generous margins to provide space for the reviewers' comments. Pages should be numbered and tables and figures may, for the purposes for review process, be within the text. The digital version of the submitted manuscript should be in MS-Word.

Authors are responsible for obtaining permission to reuse any copyrighted material contained in their articles and must provide written evidence of these permissions with their manuscript.

**The Review Process**

In general contributed manuscripts are reviewed by at least one possibly two reviewers selected by the editor. The purpose of the peer review process is to ensure that the manuscript is understandable, substantially correct, and of interest to a significant number of readers.

**General Requirements**

Some general format requirements to observe are :

1. Title Page : This page should include the main title of the manuscript and the name(s) title(s), and full mailing and contact addresses.
2. Literature Citations : All works cited in the manuscript should appear at the end of the manuscript under the heading "References". Citations within the text should be by author and year of publication, such as (Jamal 1996), or (Jamal 1996, p. 123).
3. Acknowledgements should be listed under a heading of that name, just before the "References" section.
4. Tables should be numbered and titled.
5. Artwork must be provided **separately** from the main text and in printable formats. MS-Word graphics embedded in text and other low resolution formats are not acceptable and will be returned.

# National Development Consultants

1977-2002

## KEY DATA

**FIRM NAME**
National Development Consultants Consulting Engineers and Technical Advisers.

**ABRIDGED**
NDC

**ADDRESSES**
62-M Gulberg-III, Lahore Pakistan
26-K-II, Model Town Lahore Pakistan

**PRINCIPALS**

**Engr. Barkat Ali Luna, Chairman**
☎ : 92-42 5867773, FAX: 92-42 5869287
E.mail ndc@paknet4.ptc.pk - ☎ 5880174

**Engr. Ch. Ghulam Hussain, Managing Partner**
☎ : 92-42 5864930 FAX:92-42 5862033
E.mail ndcho@lhr.paknet.com.pk -☎ 5884608

**Engr. M.A.H. Enver, Partner**
☎ : 92-42 5860870, FAX:92-42 5869287

**Engr. Ijaz Ahmad Khan, Partner**
☎: 92-42 5883729

**LANGUAGE PROFICIENCY**
English

**DESCRIPTION**
Established : 1977
Registered with Pakistan Engineering Council since 31 December 1977 under Serial No. CONSULT-71
Registered with Registrar of Firms on 12 December 1977 under Serial No. 876

**OWNERSHIP**
Partnership

**LIST OF STAFF**

**Experts**

| | |
|---|---|
| Civil Engineers | 185 |
| Agricultural Engineers /Agronomist | 16 |
| Electrical and Mechanical Engineers | 7 |
| Allied Professional (Economist, Sociologists, Soil Scientists) | 42 |
| Hydrologists, Geologist ,Hydro-geologists | 16 |
| **Support Staff** | |
| Administration and Accounts Staff | 37 |
| Technical Support Staff | 33 |
| Non-Technical Support Staff | 170 |
| **Total:** | **506** |

## ACTIVITIES

National Development Consultants give central importance to serving the Clients through sound professional advice. A flexible approach is adopted to each new assignment and emphasis placed on close cooperation and team work with the Clients. This ensures the identification of appropriate technology, project development strategy and cost controls leading to a successful conclusion of the assignment.

A commission in any of the civil engineering fields of activity may involve all or any of the services NDC offer, including :

- Investigation and Planning
- Design
- Tender Stage Assistance
- Implementation
- Operation and Maintenance

## FIELDS OF SPECIALIZATION:

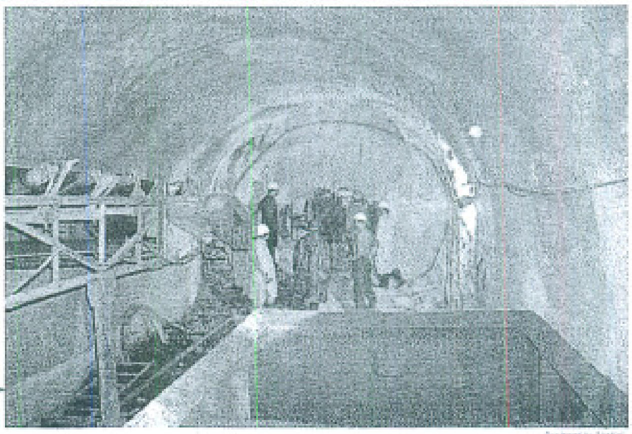
* Irrigation and Hydraulics
* Agriculture
* Drainage and Groundwater Resources Development
* River Training and Flood Protection Works
* Surveys- Topographic and Socio-Economic
* Geo-Technical Investigations
* Physical Modelling of Hydraulic Structures
* Dams and Hydro-power
* Forestry and Rural Sociology
* Project Management and Construction Supervision
* Electrical and Mechanical Operations
* Roads and Bridges
* Environmental Engineering
* Tunnels

## MAJOR ON-GOING PROJECTS

* Chashma Right Bank Irrigation Project Stage-III Client: WAPDA
* Pehur High Level Canal Project Client: Government of NWFP.
* Fordwah Eastern Sadiqia South Project Client: WAPDA
* Post Flood Rehabilitation and Protection Project
  Client: Government of Pakistan
* IIIrd Punjab On-Farm Water Management Project
  Client: Government of the Punjab
* Punjab Private Sector Groundwater Development Project
  Client: Government of the Punjab
* Remodelling Thal Canal Project Client: Government of the Punjab
* National Drainage Programme Client: WAPDA
* Second Flood Protection Sector Project-Package-B
  Client: Government of Pakistan
* Water Supply and Sewerage Projects: Clients: WASA, Lahore, Faisalabad, Multan and Sargodha.

**NDC, Certified ISO 9001:2000 is also a Member of the Association of Consulting Engineers , Pakistan (ACEP).**

**PEHUR HIGH LEVEL CANAL PROJECT (NWFP), PAKISTAN**
**Construction of Gandaf Pressure Tunnel in Progress**



Designed by Shahid