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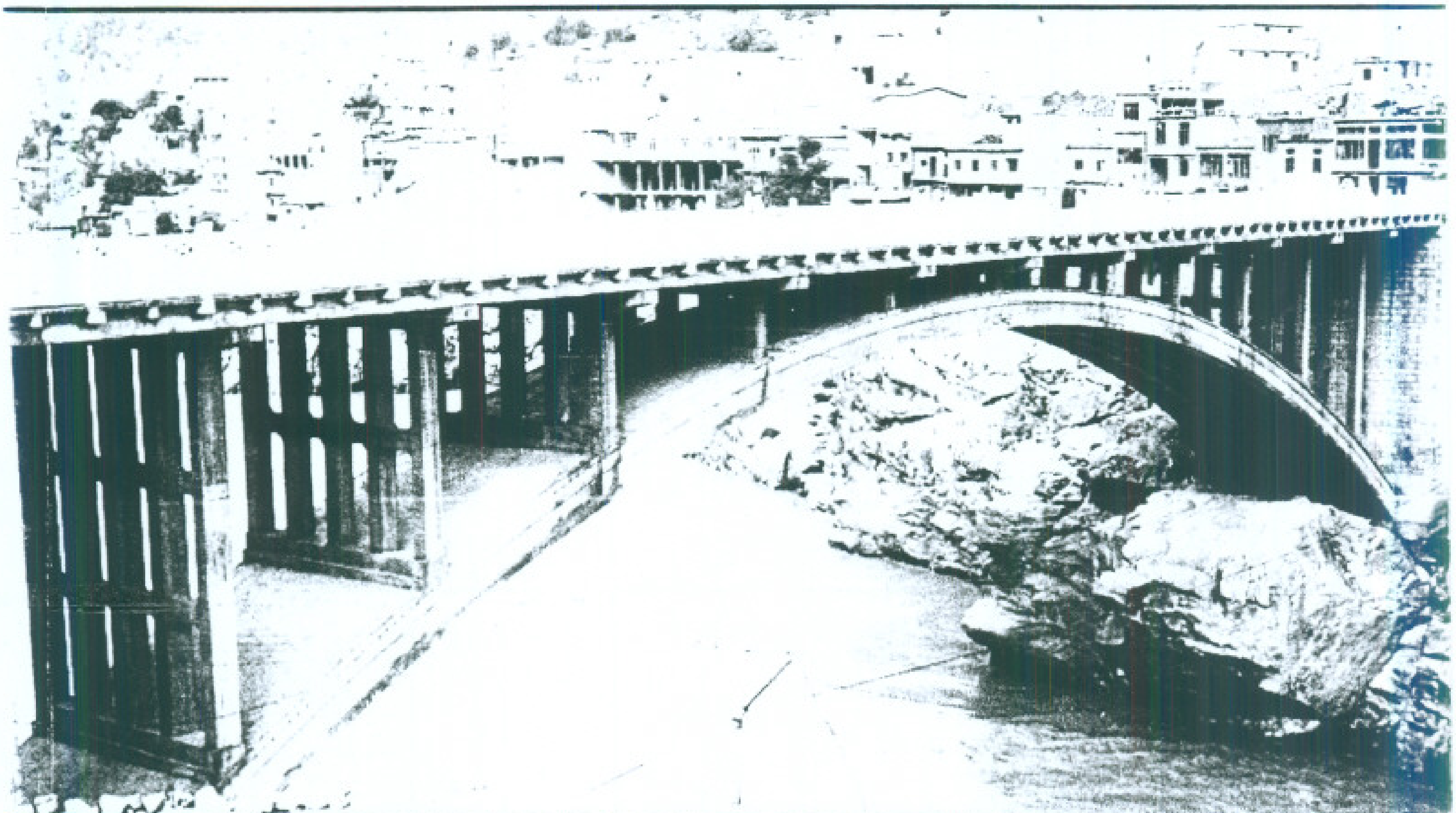

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COVER PHOTO

Upper Photo : KKH Bridge on Indus at Dassu.

Bottom Photo : Damaged KKH Bridge over a Nallah near Gilgit. Large boulders washed down stream by the rains have damaged the bridge.

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

44th YEAR OF PUBLICATION

Engineering News

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TECHNICAL VIEWS

PUBLISHED BY
Pakistan Engineering Congress

FOR MEMBERS ONLY

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EDITORIAL

BRIDGES AND THEIR UPKEEP

Bridges are some of the earliest civil engineering works and many are among the wonders of the world. Their importance for smooth and uninterrupted flow of the present day fast moving traffic can not be denied. Bridges/culverts are a very costly component of any road. Consequently these require much more attention as compared to other components of the road due to huge investment in their construction. It has been commonly seen that in poor countries, unfortunately the bridges once constructed are left on their own and very little budgetary provision is made for their regular upkeep/maintenance. The regular maintenance staff also pays a very little attention to this aspect of road maintenance. In fact there is no culture of regular inspection, upkeep and maintenance of bridges in the country.

One only has to drive along any road/highway any where in the country to see bridges/culverts with missing side rails or parapet walls and in some cases gapping holes in the deck which are nothing but death traps.

Another commonly observed undesirable practice is that the river/nullah beds are encroached by human settlements, industries etc. restricting waterway for the flow of water through the bridges/culverts. This practice endangers the structure as well as human life and property due to spilling over the banks and flooding of the adjoining areas.

What we need is a mandatory annual general visual inspection programme of all bridges/culverts to check the health of the structure for safety and in the case of major structures there should be more elaborate programme of monitoring with the help of modern instruments/techniques now available. The inspection programmes already prescribed by some Departments/Authorities are neither elaborate nor are these regularly followed. It is high time to introduce regular and elaborate inspection programmes for bridges/culverts with emphasis on preventive maintenance to make best use of the huge investment and protect public life and property. It would be desirable to dedicate some competent and trained staff exclusively for this purpose. This will go a long way in improving safety record and shall save on the huge costs of rehabilitation/reconstruction of bridges.

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 Fold.

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1 Engr. Shahzad Zeb Khan
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3 Engr. Najum-ul-Ghafoor
4 Engr. Anees Ahmad
5 Engr. Zaheer Ahmad Awan
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7 Engr. Muhammad Yasir

Members admitted on 25-1-2003

1 Engr. Brig. Syed Abrar.Hussain
2 Engr. Abid Alla-ud-Din Qureshi

Members admitted on 8-3-2002

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2 Engr. Sabir Ali
3 Engr. Maraj Khalid
4 Engr. Miss Rubina Shehzadi

Members admitted on 29-3-2003

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12 Engr. Muhammad Khalil

گذشتہ اور نئی صدی کے طویل در(SPAN) والے دو معلق پُل
(SUSPENSION BRIDGES)

اس مضمون میں طویل در والے دو معلق پُلوں کا ذکر ہے۔ پہلا اکاشی کیکیو پُل (Akashi-Kaikyo Bridge) جاپان میں ہے اور آرٹیکل نمبر 1 میں مذکور ہے۔ دوسرا رنیانگ جنوبی پُل (Runyang South Bridge) جو چین میں ہے اور آرٹیکل نمبر 2 میں مذکور ہے۔

اکاشی پُل 1998ء میں تعمیر ہوا تھا۔ 1991 میٹر طویل بڑے در کی بدولت یہ گذشتہ صدی کا عظیم ترین اور دُنیا میں اِس طرز کے تعمیر کئے گئے پُلوں میں تاحال طویل ترین در والا پُل ہے۔

1490 میٹر طویل رنیانگ پُل کی تعمیر 2000ء میں شروع ہوئی جس کی تکمیل 2005ء میں متوقع ہے۔ تکمیل کے بعد اس پُل کا شمار دنیا کے اس طرز کے طویل ترین در والے پُلوں میں تیسرے نمبر پر ہوگا۔ نیز نئی صدی کی ابتداء میں تعمیر ہونے والا یہ عظیم ترین پُل ہوگا۔

مضمون میں برسبیل تذکرہ ایک مختلف طرز کے پُل کا بھی ذکر کیا گیا ہے۔ یہ ہانشو (Honshu) جزیرے میں واقع سُوکوہارا (Sukuhara) پُل ہے جو اکاشی پُل سے چند کلومیٹر کے فاصلے پر ہے۔ سُوکوہارا پُل دو متصل پُلوں پر مشتمل ہے۔ 1998ء میں تکمیل کے وقت پری سٹریسڈ کنکریٹ (Prestressed Concrete) سے تعمیر شدہ 180 میٹر طویل در کی بدولت یہ دنیا میں اس طرز کے بنائے گئے پُلوں میں طویل ترین در والا پُل تھا۔ تاہم اس پُل کی تعمیر کے بعد جلد ہی دنیا میں اِس کی نسبت طویل در والے اِسی طرز کے دیگر پُل تعمیر ہوئے۔ جاپان میں تعمیر کئے گئے ایسے دو پُلوں میں سے ایک کیسو پُل (Kiso Bridge) ہے جس کے تین بڑے دروں میں سے ہر ایک 275 میٹر طویل ہے۔ دوسرا اِبی پُل (Ibi Bridge) ہے جس کے چاروں دروں میں سے ہر ایک 271.50 میٹر طویل ہے۔

LONG-SPAN CABLE-SUPPORTED BRIDGES

by

Juhani VIROLA, Eur Ing-FEANI

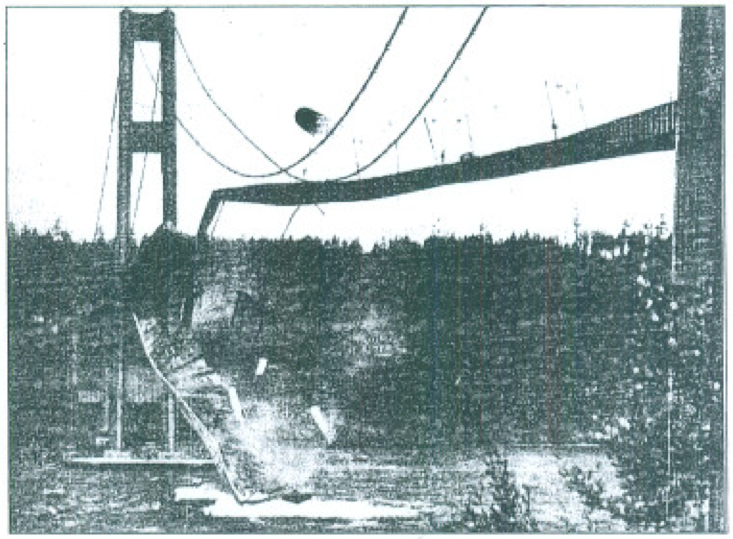
Teemuaho Group Helsinki, Finland

1. ABSTRACT

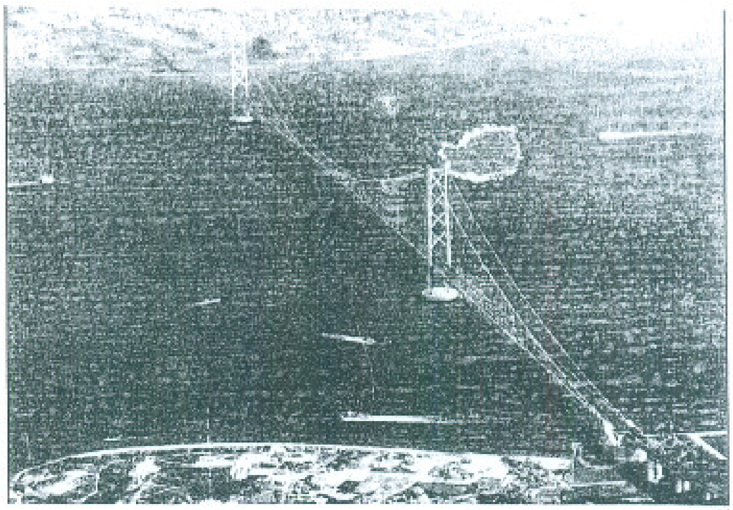
It has been said that "Temples and bridges are the most outstanding exhibits of mankind". On the other hand, never earlier have so many notable (long-span) bridges been built as during the change of the past Millennium. The boom of great bridges concerns long-span suspension bridges, and particularly cable-stayed bridges.

In turn of the Millennium, there are existing in the world 17 bridges with a span longer than 1000 m and completed before 2000. These all are suspension bridges (Table 1). Counted by countries : 4 in the U.S.A. and Japan; 2 in Britain, Turkey and China; 1 in Portugal, Sweden and Denmark.

Hitherto, suspension bridge is the only bridge type exceeding the one kilometre span. As appears from Table 1, the span of suspension bridges is already nearing the 2 km mark. Further to the development of suspension bridges with respect to the growth of the main span, in this article the materials of towers are discussed, and the methods of assembling main cables and the increase of tensile strength of the cable-wires are outlined. Also the two main types introduced for the deck structures are described, particularly after the collapse of the first Tacoma Bridge (in 1940) (Picture 1): strong steel truss stiffening girders (USA) and novel streamlined steel box girders (Europe and Asia). The importance of resistance against earthquakes is pointed out, particularly in view of the survival of the Akashi Bridge in the notorious Kobe Earthquake 1995.



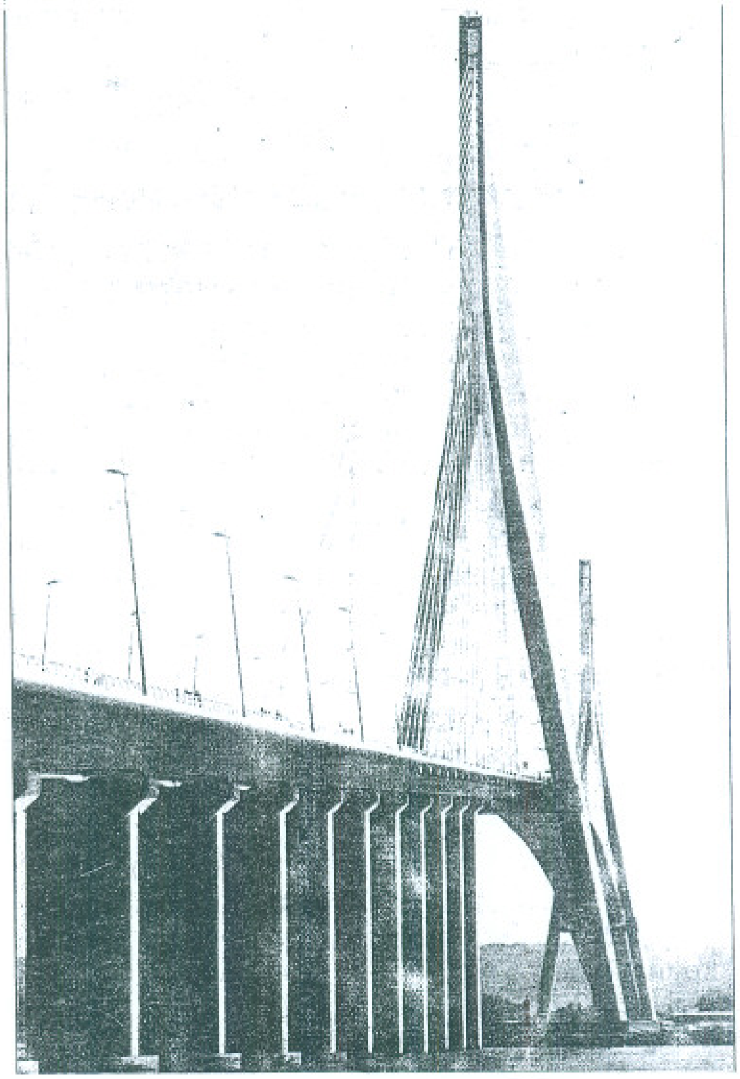
1. First Tacoma Bridge, Washington State, collapsed in a storm only four months after completion.
PHOTO WASHINGTION STATE DEPARTMENT OF HIGHWAYS



2. Akashi-Kaikyo Bridge, Japan, record bridge in many respects, among them the longest main span of any bridges.
PHOTO HONSHU-SHIKOKU BRIDGE AUTHORITY

Among suspension bridges, the No.1 is the mighty Akashi-Kaikyo Bridge (Picture 2), main span 1991 m, completed in 1998. It is situated near Kobe, in Japan, along the Kobe-Naruto Route, between Honshu and Shikoku Islands. The No.2 is the Great Belt East Bridge in Denmark (1624 m; 1998). Among suspension bridges for highway/railway traffic, the No.1 is the Tsing Ma Bridge in Hong Kong, China (1377 m ; 1997) [5].

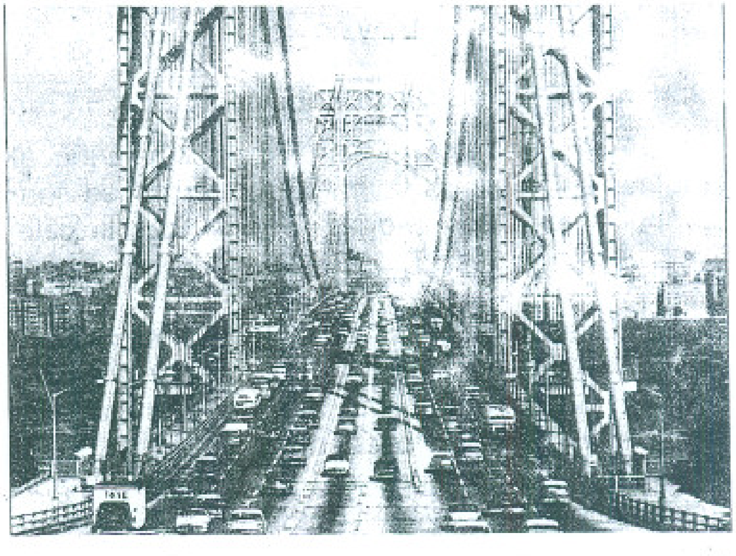
Cable-stayed bridges are now approaching the 1 km span limit. The No.1 (Picture 3) is the Tatara Bridge in Japan (span 890 m, dated 1999), and the No.2 is the Pont de Normandie in France (856 m; 1995). The projected Stonecutters Bridge in Hong Kong, China, is to be the first cable-stayed bridge in the world to exceed the 1 km mark (1018 m; 2008). Among cable-stayed bridges for highway/railway traffic, the No.1 is the Oresund Bridge between Sweden and Denmark (490 m ; 2000).



3. Port de Normandie, France, currently the world's longest-span cable-stayed bridge.
PHOTO C. LOUVET

Other bridge types than cable-supported bridges may reach spans of 300-500 m (Table 3). Successive main span records during the past two Millennia are given in Table 4.

At the end of article some proposed long-span bridges of the future are mentioned, for instance suspension bridges like Qiongzhou in China (span 2000 m to 2500 m) [6]; Messina in Italy (3300 m); Tsugaru in Japan (4000 m) [7]; Gibraltar between Spain and Morocco (5000 m). Also, a gigantic cable-stayed bridge was proposed in the 1980's across the Gibraltar Strait with an enormous main span of 8400 m [11].



4. George Washington Bridge, New York City, almost doubled the previous span length record.
PHOTO THE PORT AUTHORITY OF NY & NJ

2. BRIDGES OF THE 1930's

In the 1930's, two unprecedentedly large suspension bridges were built, both in the USA, in chronological order: George Washington (Picture 4) (span 1067 m) and

Golden Gate (1280 m), while in the 1940's no bridge of this magnitude was built anywhere.

2.1 The first bridge surpassing one kilometre span (New York)

At the change of the 1800/1900's it seemed possible that cantilever steel truss girder bridges could compete with suspension bridges in span lengths (Table 3).

However, in the early 1930's suspension bridge re-conquered its position as the leading long-span bridge type. In Oct.1931, the mighty George Washington Bridge in New York City across the Hudson River was opened to traffic. It was the first bridge surpassing the one kilometre span, and with its span of 1067 m it improved the previous record by as much as 89 % (Table 3). For a start, the bridge had 6 traffic lanes. Since 1962, when the lower level of the deck was opened to traffic, the bridge carries 14 lanes. The annual number of vehicles (year 1991) was 100 mill., which is more than for any other long-span suspension bridge. The cross-section of the steel truss deck measures 32.3 x 9.1 m, and the underclearance is 65 m (in this article the term "underclearance" means the navigation clearance or vertical clearance at mid-span). The bridge has 4 cables, each 90 cm in diameter. The towers, 180 m in height, are of open steel truss construction. There was a plan to cover afterwards the towers with concrete, but this was not realized.

Most of the major suspension bridges (span >1000 m) have 2 main cables, while the G.Washington and Verrazano bridges have 4 cables The steel wires in the cables of the G.Washington Bridge have an ultimate tensile strength of 1550 MPa (155 kp/mm2).

Generally, in 1830 the wire strength was only 600 MPa, in the 1880's 1100 MPa (Brooklyn), in the 1930's 1550 MPa (G.Washington), and in the late 1990's 1800 MPa (Akashi), so during 170 years it tripled. In major suspension bridges (span >1000 m), the main cables have a circular cross-section accommodating thousands of high-strength steel wires of ca. 5 mm in diameter, while in shorter suspension bridges also other arrangements may be used.

2.2 The world's most famous bridge (San Francisco)

The well known San Francisco earthquake occurred in 1906. Another notable earthquake took place in 1989. It damaged some buildings and bridges, however, not proper suspension bridges. Due to their flexible structure, suspension bridges usually can withstand earthquakes better than other bridge types.

The majestic Golden Gate Bridge is situated at the entrance of the San Francisco Bay. The bridge was opened to traffic in May 1937. Its 1280 m span was longest in the world until 1964 (completion of the Verrazano Bridge). The bridge has six traffic lanes, and pedestrian/bicycle lanes at both sides. The steel truss deck measures 27.4 x 7.6 m, and the underclearance is 67 m. The towers are 227 m, highest in the world until 1998 (completion of the Akashi Bridge). The two cables measure 90 cm in diameter.

The proportions of the Golden Gate Bridge are graceful. This red-coloured beauty is undoubtedly the best-known bridge in the world. The Golden Gate Bridge survived the earthquake in 1989. A plan to increase the earthquake resistance of the bridge is underway.

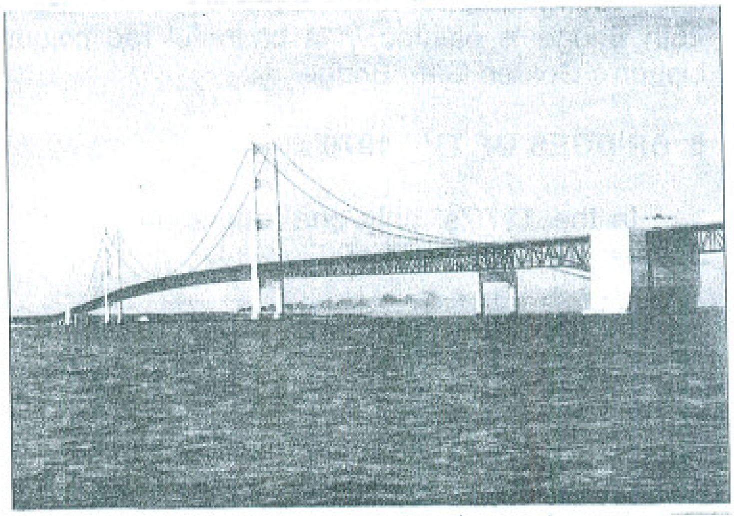
3. BRIDGES OF THE 1950's

No major bridge (span >1000 m) was built in the 1940's. The well known collapse in 1940 of the first Tacoma Bridge (span 853 m; dated 1940) led to an opposite extremity. From there on, rigid suspension bridges were constructed by using a heavy stiffening truss deck, in some cases may be even too heavy, particularly in the USA. In the 1950's, one major suspension bridge (>1000 m) was built in the world: the Mackinac Bridge (1158 m).

3.1 A strong bridge in Michigan

The most remarkable of the bridges constructed in the 1950's is the one across the Mackinac Straits in Michigan, between Lake Michigan and Lake Huron, with an overall length of 5,8 km. The Mackinac Bridge was opened to traffic in Nov.1957, at which time its span of 1158 m ranked 2nd in the world, directly after the Golden Gate Bridge. The two 549 m side spans are notably long. The steel towers are 168 m tall.

The Mackinac Bridge (Picture 5) is claimed to be the world's strongest suspension bridge, according to one estimate supposed to withstand even winds of 400 km/h. This is achieved by means of some structural arrangements. For instance, the two middle lanes of 4 lanes in all are of open grid construction (the same arrangement is applied to the Lisbon Bridge, mentioned below). The strong steel truss deck measures 20.7 m x 11.6 m, and the underclearance is 45 m. On the bridge site, winter climate is severe.



5. Mackinac Straits Bridge, Michigan, claimed to withstand winds of 1000 km/h.
PHOTO MACKINAC BRIDGE AUTHORITY

4. BRIDGES OF THE 1960's

In the 1960's, 3 long-span (> 1000 m) bridges were completed in the world: Forth (1006 m), Verrazano (1298 m), and Salazar, later Ponte 25 de Abril (1013 m), while the 4th one, the novel Severn Bridge in Britain is narrowly below the one kilometre mark (span 988 m ; completed 1966).

4.1 The first major suspension bridge in Europe (Edinburgh)

In Sept.1964, the Firth of Forth Road Bridge was opened to traffic near Edinburgh, Scotland. Its span is 1100 yards or 1006 m, and this was first time in Europe when the one kilometre span was exceeded. The steel towers are 156 m high. The light steel truss deck is 23.8 m wide, totaling 36.3 m including the cantilevers for light traffic, and 8.4 m deep. The underclearance is 45 m. A plan is underway to strengthen the bridge for heavier traffic.

4.2 Another record bridge in New York

The Verrazano-Narrows Bridge is the most remarkable bridge in the 1960's. It was completed in Nov.1964 at the entrance of the Port of New York. With a span of 1298 m it was the leading long-span suspension bridge until the Humber Bridge was completed in 1981. The steel towers of the Verrazano bridge are 210 m tall. The bridge has 4 cables, each 90 cm in diameter, and two traffic levels totaling 12 lanes. The cross-section of the steel truss deck measures 31.4 x 7.3 m, and the underclearance is 69 m. In 1991, annual traffic volume reached 70 mill. vehicles.

4.3 A beautiful red bridge at Lisbon

Lisbon's first bridge across the Tagus River was completed in Aug.1966, at which time it was named Ponte Salazar. During the April 1974 revolution, the bridge was renamed Ponte 25 de Abril. The span of the bridge, 1013 m, was the longest in Europe until 1973, when the Bosporus Bridge was completed. The steel towers are 190 m tall, tallest suspension bridge towers in Europe until completion of the Great Belt East Bridge in 1998. The foundation depth of the south tower of the Lisbon Bridge is 80 m below water level, while the bridge's underclearance is 70 m. These two dimensions are still records for long-span suspension bridges. The steel truss deck measures 21.0 x 10.7 m.

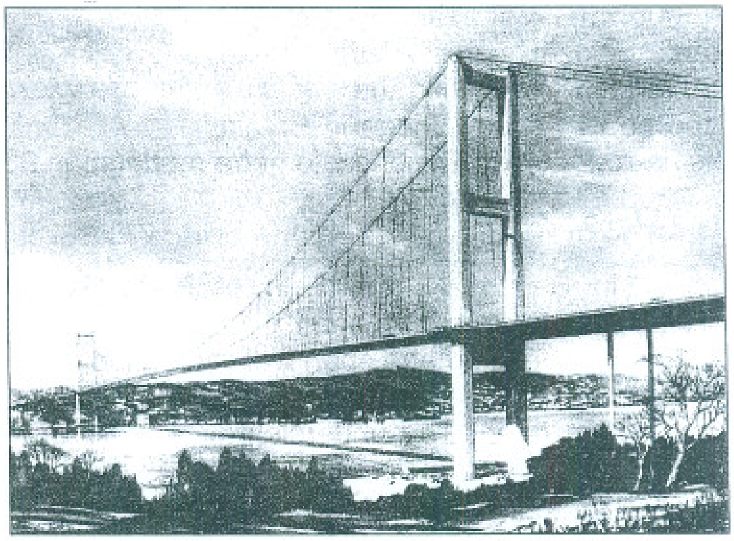
A project to strengthen the bridge was performed in the 1990's: to add two extra main cables above the two existing ones, add a railway onto the lower level of the deck, and widen the upper level of the deck to accommodate 6 traffic lanes in all. The April 25th Bridge is painted in a beautiful red colour, and sometimes it is referred to as Lisbon's Golden Gate Bridge.

5. BRIDGES OF THE 1970's

In the 1970's, only one long-span (>1000 m) bridge was built in the world: the Bosporus Bridge (1074 m).

5.1 The first intercontinental bridge (Istanbul)

The Bosporus Bridge (Picture 6) was opened to traffic in Oct.1973 in Istanbul. With its 1074 m span it was Europe's leading long-span bridge (until completion of the Humber Bridge in 1981) and the first intercontinental bridge from Europe to Asia. The Bosporus Bridge borrows its features from the above mentioned Severn Bridge : inclined hangers and a streamlined steel box girder deck. The deck is 3.0 m deep, 28.0 m wide, and the total width is 33.4 m, including the walkways at each outside edge for maintenance crew access. The steel towers are 165 m tall. Only the main span is suspended, while the side spans are unsuspended.



6. Bosporus Bridge, Istanbul, a three mellenniums dream of a bridge between Europe and Asia.
PHOTO KARAYOLLARI GENEL MUDURLUGU

In Table 1, the following bridges (span >1000 m) have a steel box girder deck: Great Belt, Runyang, Humber, Jiangyin, Tsing Ma, Höga Kusten, Fatih Sultan Mehmet, Bosporus, Kurushima-3&2, while the others (>1000 m) have a steel truss girder deck.

6. BRIDGES OF THE 1980's

In the 1980's, 3 long-span bridges (>1000 m) were completed: Humber (1410 m), Fatih Sultan Mehmet (1090 m), and Minami Bisan-seto (1100 m). The 4th one, the Kita Bisan-seto Bridge has a shared cable anchorage with the Minami Bridge and misses the one kilometre mark only by 10 m (990 m; 1988).

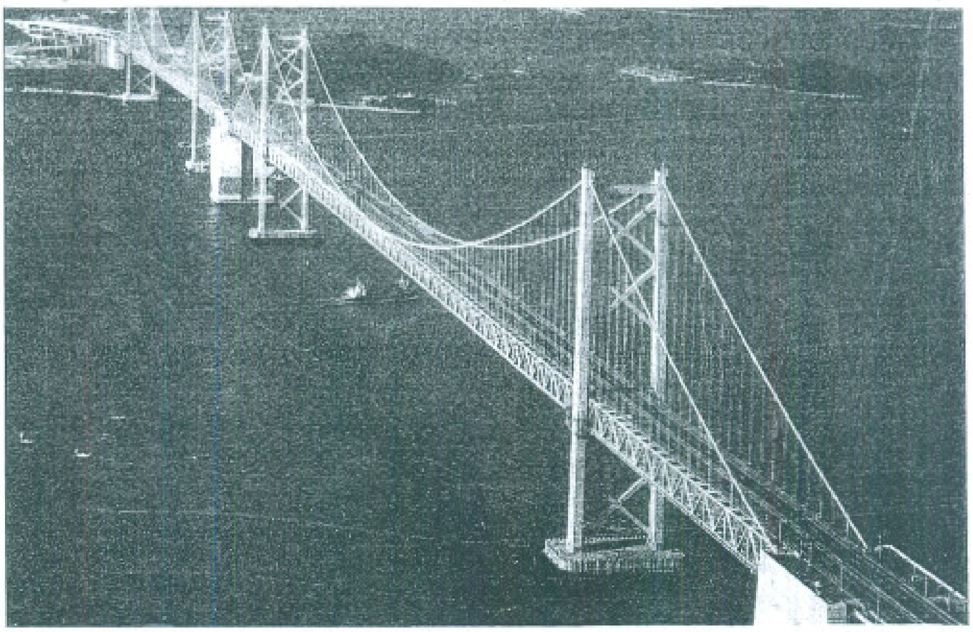
6.1 The record moves again to Europe (England)

The bridge across the Humber Estuary at Kingston-upon-Hull, Britain, is the largest suspension bridge in the 1980's. It was opened to traffic in July 1981. The 1410 m span of the Humber Bridge returned the record from America to Europe, first time since 1890. It was the world's leading long-span bridge until completion of the Akashi Bridge in 1998. The deck of the Humber Bridge is 4.5 m deep (50% deeper than Severn and Bosporus bridges), 22.0 m wide, and 28.5 m wide in all including the cantilevers for light traffic.

The Humber Bridge is a unique long-span suspension bridge. It exposes the following three technical features: concrete towers (163 m tall), inclined hangers, and a streamlined steel box girder deck. The Humber Bridge is the last major suspension bridge (span >1000 m) constructed with inclined hangers. In bridges built later, vertical hangers are used again. The notably unequal length of the two suspended side spans, 280 m and 530 m, is also a feature of the Humber Bridge. The underclearance is notably low, 30.0 m.

The Humber Bridge is the first bridge with a span >1000 m having concrete towers. In Table 1, the following bridges (span >1000 m) have concrete towers: Great Belt, Runyang, Humber, Jiangyin, Tsing Ma and Höga Kusten, while the others (>1000 m) have steel towers.

6.2 The world's longest road/rail bridge (Japan)



7. Seto-Ohashi Succession of Bridges, Japan, currently the world's longest higway/railway bridge.
PHOTO HONSHU-SHIKOU BRIDGE AUTHORITY

In April 1988, a long succession of bridges was completed between Honshu and Shikoku Islands, along the Kojima-Sakaide Route in Japan. This includes some remarkable cable-stayed and suspension bridges, and it is called the Seto-Ohashi Bridge (Picture 7). The overall length of this bridge combination, 12 km 306 m, makes it the world's longest highway/railway bridge.

There are three large suspension bridges on this route: Minami Bisan-seto (span 1100 m), Kita Bisan-seto (990 m), and Shimotsui-seto (940 m). The Minami and Kita bridges have three normal suspension spans. In the Shimotsui Bridge only the main span is suspended, while the side spans are unsuspended. These all three bridges have steel towers and 2-level steel truss decks.

With its 1100 m span the Minami Bridge was the world's leading long-span highway/railway bridge, until completion of the Tsing Ma Bridge in 1997. The steel towers of the Minami Bridge are of different height, 194 m and 186 m. The steel truss deck measures 30.0 x 13,0 m, and the underclearance is 65 m. The bridge has two cables 1070 mm in diameter, and this is the first suspension bridge to have cables larger than 1000 mm.

In the Minami and Kita bridges, a modern prefabricated parallel-wire strand (PPWS) technique was applied. In case of the Minami Bridge, this technique was first time used for such a large bridge (span >1000 m).

In cable construction of major suspension bridges (Table 1), the PPWS technique is applied in some bridges in Japan (Akashi, Minami, Kurushima, Kita) and in China (Runyang, Jiangyin, Xiling, Tiger Gate, probably Yichang, but not Tsing Ma), which two countries have factories needed for this technique. In most of the bridges in Table 1, AS-technique (Air Spinning) is used.

6.3 Another intercontinental bridge (Istanbul)

In July 1988, the second suspension bridge was completed across the Bosporus in Istanbul. It is located some kilometres upstream from the first bridge. The second crossing is called the Fatih Sultan Mehmet Bridge. It has a span of 1090 m, a few metres longer than the first bridge. Similarly to the first bridge, the second bridge also has a streamlined steel box girder deck, but contrary to the first bridge the second bridge has vertical hangers. The second bridge has 4 + 4 traffic lanes, while the first bridge has 3 + 3 lanes. The deck of the second bridge is 3.0 m deep, 33.8 m wide, and 39.4 m wide including walkways at each outside edge for maintenance crew access. At completion, it was the widest long-span suspension bridge in the world, until completion of the Tsing Ma Bridge in 1997.

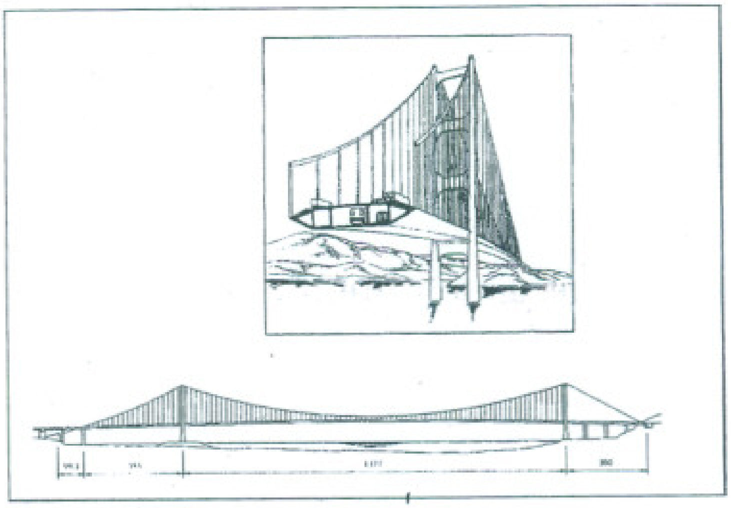
The two Bosporus bridges are equipped with pedestrian/bicycle paths on both sides, though not in use (nominally for safety reasons to the users). Both bridges have an underclearance of 64 m. The second bridge is situated between steep river banks. It has no side spans at all, thus it expressly is a single-span suspension bridge. The towers rise 100 m high above the piers, and 165 m above the water level.

7. BRIDGES OF THE 1990's

In the 1990's, 7 long-span (>1000 m) bridges were completed, more than during any previous decade: Tsing Ma (1377 m), Höga Kusten 1210 m), Akashi (1991 m), Great Belt (1624 m), Kurushima-3 & 2 (1030 m and 1020 m), and Jiangyin (1385 m).

7.1 The world's leading long-span road/rail bridge (China)

In Hong Kong, the Tsing Ma Bridge (Picture 8) was built during 1992-1997 and opened to traffic in Apr.1997, a few months before Hong Kong was returned to China in July 1997. The bridge leads to the new Chek Lap Kok Airport, and with its 1377 m span it currently is the world's leading long-span highway/railway bridge. The longer side span, 359 m, is suspended, while the shorter side span, 300 m, is unsuspended.

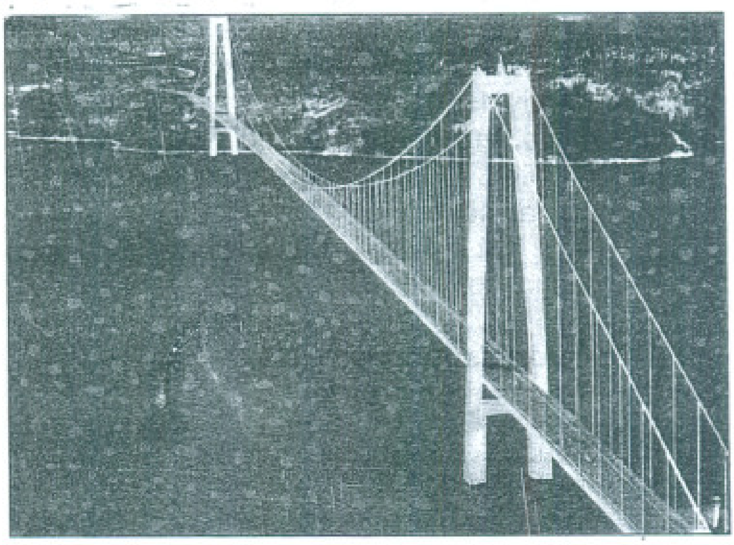


8. Tsing Ma Bridge, Hong Kong, the world's longest-span highway/railway bridge.
PHOTO CONSTRUCTION TODAY

The bridge has two cables 1100 mm in diameter. The deck, 41.0 x 7.2 m, is a combination of a steel truss and a steel box girder construction. At completion, it is the widest long-span suspension bridge in the world. The upper level accommodates 6 lanes for highway traffic. The railway traffic runs along the lower level inside the box, where there are also two emergency lanes for road traffic, to be used in case of violent typhoons. The underclearance is 62 m, and the concrete towers are 206 m tall.

7.2 The world's northernmost long-span bridge (Sweden)

During 1993-1997, a notable bridge was built across the Ångerman River in Sweden and inaugurated in Dec.1997. It is called the Höga Kusten Bridge (Picture 9), and its span of 1210 m ranks directly after the famous Golden Gate Bridge. The Höga Kusten Bridge has concrete towers, 180 m in height, a streamlined steel box girder deck, 22.0 x 4.0 m in cross-section. The underclearance is 40 m. For a start the bridge had 2 traffic lanes and 2 sidewalks, now 4 traffic lanes. For aesthetic reasons, also the side spans are suspended.



9. Hoga Kusten Bridge, Sweden, 8th with respect to span length at the change of the millennium.
PHOTO VAGVERKET REGION MITT

7.3 A gigantic bridge in the middle of earthquake (Japan)

During 1988-1998, a gigantic suspension bridge was built across the Akashi Straits in Japan. The bridge is situated on the Kobe-Naruto Route, between Honshu and Shikoku Islands, south-west of Kobe City. The 1991 m span of the Akashi-Kaikyo Bridge is the first one surpassing besides an English mile (1609 m) also a nautical mile (1852 m). The two side spans are enormously long, 960 m each, the longest suspended side spans in the world. The steel towers are 297 m high, tallest bridge towers in the world. The steel truss deck accommodates 6 lanes for highway traffic, and the underclearance is 65 m. The deck measures 35.5 x 14.0 m in cross-section, and the deck is deeper than in any other long-span suspension bridge.

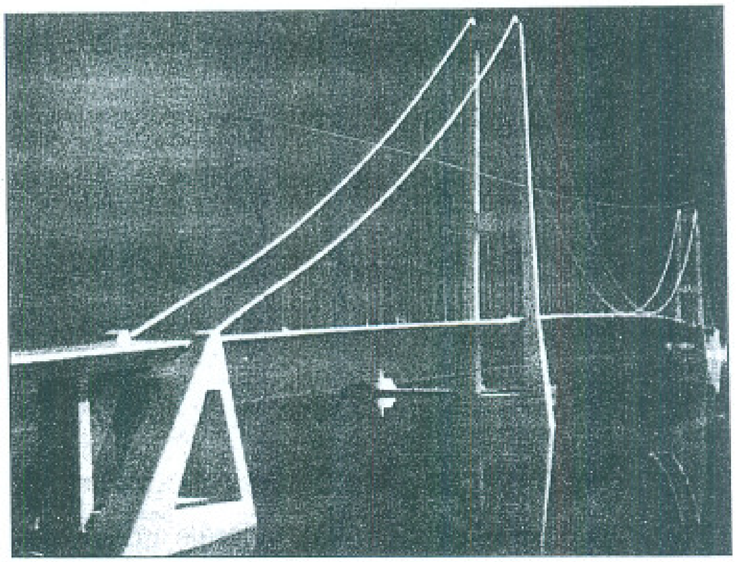
Originally there was a plan to accommodate a railway onto the lower level of the deck, but this plan was abandoned. The railway load would have required 4 cables, while two cables are sufficient for the road traffic. The two cables are 1122 mm in diameter, which is a record. The steel wires of the cables have an ultimate tensile strength of 1800 MPa, also a record. The cables were built of prefabricated paralled-wire strands (PPWS). Consequently, this technique is suitable even for suspension bridges of this magnitude. The pilot wire was carried across the straits by a helicopter, a technique applied for the first time. The atmosphere environment inside the cables is improved by injecting dried air into the cables, a method used first time for a major suspension bridge.

In January 1995, an earthquake of 7.2 on the Richter scale occurred in Kobe area, and more than 5000 people were killed. The epicentre of the earthquake was located at the south end of the Akashi Bridge, several kilometres away from Kobe City itself. The bridge experienced a severe earthquake resistance test, as the magnitude at the bridge site was nearly 8 on the Richter scale. When the earthquake occurred, the towers and the cables of the bridge were newly completed, and the construction of the deck was to be commenced. Buildings and bridges collapsed even at a distance of 50 km from the Akashi Bridge.

According to a preliminary research, the Akashi Bridge having a design load of 8.5 on the Richter scale earthquakes survived with only minor damages. Because of the movement of the earth the distance between the foundations of the towers increased 80 cm and the tops of the towers inclined 10 cm. The main span increased 80 cm and became nearly 1991 m, and as a result the sag of the cables decreased 130 cm. It is estimated that the damages would not have been this slight had the cables not been already installed. Despite the earthquake, the Akashi Bridge was completed according to the original schedule in Spring 1998, and it was officially inaugurated in April 1998.

7.4 The world's longest continuous steel box girder (Denmark)

In Denmark, the construction of the Great Belt Suspension Bridge (Picture 10) was commenced in 1991. At completion in June 1998, its 1624 m long span ranks the 2nd in the world. It also has the world's tallest bridge towers made of concrete, 254 m. The streamlined steel box girder deck, 31.0 x 4.0 m, is continuous beyond the towers at a length of 2694 m, from anchorage



10. Great Belt Bridge, Denmark, for a short period the world's longest-span suspension bridge.
PHOTO STOREBAELTSFORBINDELSEN

to anchorage. This is the longest continuous steel girder bridge deck in the world (the three spans of the Akashi Bridge total 3911 m, but its deck is not continuous at the towers). There was a dispute between Finland and Denmark concerning the underclearance of the bridge, 65 m, but in Autumn 1992 an agreement was reached.

7.5 A unique triple suspension bridge (Japan)

Between Honshu and Shikoku Islands, on the Onomichi-Imabari Route in Japan, the Kurushima-Kaikyo Bridge was opened to traffic in May 1999. It comprises three successive suspension bridges, with two shared cable anchorages. The Kurushima bridges have streamlined steel box girder decks. The Kurushima-1 Bridge is the smallest one. It has a main span of 600 m and its all three spans are suspended. The Kurushima-2 Bridge is in the middle. It has a main span of 1020 m. The main span and one of the side spans are suspended.

The Kurushima-3 Bridge is the largest of these three suspension bridges. Its main span is 1030 m long, and only that is suspended. The alignment of outermost side span of the Kurushima-3 Bridge is curved and the cables descend beyond the side span down to the end anchorage. The steel box girder deck at main span of the Kurushima-3 Bridge measures 32,3 x 4,3 m, and the underclearance is 65 m. The steel towers are 184 m tall.

7.6 The past millennium's last major suspension bridge (China)

In Jiangsu Province in China, the great Jiangyin Bridge over the Yangzi River was inaugurated in Sept.1999. At completion its 1385 m span ranks 4th in the world. Only the main span is suspended, the two approach spans are not. The bridge has concrete towers, 196 m in height. The main span consists of a streamlined orthotropic steel box girder, depth 3.0 m, while the approach spans are of concrete. The width of the deck is 32.5 m between hangers, accommodating 6 traffic lanes, and the overall width is 36.9 m, with a 2.2 m walkway at each outside edge for maintenance access. The underclearance is 50 m.

8. NEW MILLENNIUM

In the early years of the new millennium, several long-span suspension bridges are under design. The first of these, already under construction, is the Runyang South Bridge (span 1490 m, 3rd longest in the world).

8.1 The new millennium's first major suspension bridge (China)

In China, a large bridge complex is under construction across the Yangzi River in Jiangsu Province, downstream of Nanjing. Due to the island of Siyezhou in the river, the crossing consists of 2 major bridges which will link Zhenjiang on the south bank of the river and Yangzhou on the north. The south bridge is a suspension bridge with a main span of 1490 m, while the north one is a cable-stayed bridge with a main span of 406 m. The bridge complex is known as the Runyang Bridge [4]. The new connection will form an important link in the Beijing-Shanghai Expressway. Construction of the Runyang Bridge was commenced in Oct. 2000, due for completion in 5 years by Oct. 2005.

With its main span of 1490 m the suspension bridge will rank as 3rd in the world ever built. Only its main span is suspended, the two approach spans are not. The bridge has concrete towers, 215 m in height. The main span consists of a streamlined orthotropic steel box girder, depth 3.0 m. The width of the deck of the main span is

34.3m between hangers, accommodating 6 traffic lanes, and the overall width is 39.2 m, with a narrow m walkway at each outside edge for maintenance access. The underclearance is about 50 m (measured from drawing).

9. CABLE-STAYED BRIDGES

Cable-stayed bridges are now approaching the 1 km span limit. In change of the millennium, there are in the world >10 cable-stayed bridges with a span >500 m, completed or under construction (Table 2).

The No.1 is the Tatara Bridge, main span 890 m, dated May 1999. It is situated in Japan along the Onomichi-Imabari Route, between Honshu and Shikoku Islands, some 200 km west of Kobe. The No.2 is the Pont de Normandie in France (856 m; 1995). The Stonecutter Bridge in Hong Kong, China, may be the first cable-stayed bridge in the world exceeding the 1000 m span limit (1018 m; to be built during 2003-2008) [5].

Among cable-stayed bridges for highway/railway traffic, the No.1 is the Oresund Bridge between Sweden and Denmark (490 m; 2000). The No. 2 is the Kap Shui Mun Bridge in Hong Kong, China (430 m; 1997), close to the Tsing Ma Bridge mentioned ahead [5].

Among 1-tower cable-stayed bridges, the No.1 is the Surgut Bridge in Russia (408 m; 2000). The projected Ulyanovsk Bridge, also in Russia, was aimed to have 1 tower and 2 main spans of 407 m each, but this plan is under modification.

10. LONG-SPAN BRIDGES OF THE FUTURE

In the beginning of the new millennium, plenty of long-span bridges are under consideration worldwide. To mention some of those :

In China, several long-span suspension bridges are planned or are under construction, for instance Tsing Lung (span 1418 m) and Lingdingyang (1450 m) [5]; Jiaozhouwan (1652 m or 1800 m) [4] ; Qiongzhou (2000 m or 2500 m) [6].

In Turkey, the projected Izmit Bay suspension bridge (span 1668 m) was to be commenced in change of the millennium, but the destructive Izmit earthquake in 1999 postponed the commencement. An earlier proposal was a hybrid cable-stayed and suspension bridge (span 2000 m). Also in Turkey, the Chanakkale suspension bridge (span 1510 m) is under consideration across the Dardanelles.

Between Denmark and Germany, various proposals have been made to build a bridge/tunnel across the Fehmarn Belt. One of those is a suspension bridge with a 1752 m span. In Indonesia, a long-span (2300 m) hybrid cable-stayed and suspension bridge is under consideration between the islands of Java and Bali.

In Japan, a gigantic bridge across the Tsugaru Strait between Honshu and Hokkaido Islands is under consideration. One proposal is a hybrid of suspension and cable-stayed bridges, two successive main spans 4000 m each and two 2000 m long side spans [7]. Since 1988, there already is existing the Seikan Tunnel for railway traffic under the Tsugaru Strait. It is 53.8 km long, currently the world's longest railway tunnel, a few kilometres longer than the Eurotunnel under the English Channel.

During the past few decades, various proposals have been made to build a bridge/tunnel across the Messina Straits between Sicily and Italy. A recent plan suggests a suspension bridge with a main span of 3300 m.

Similarly, between Spain and Morocco across the Gibraltar Strait, various bridge/tunnel alternatives have been proposed. One plan suggests an enormous suspension bridge with two successive main spans 5000 m each and two 2000 m side spans.

To mention briefly some other suspension bridges under consideration, span > 2000 m [8] :

* Hoyo Strait (B), Japan, 3000 m
* Hoyo Strait (A), Japan, 2800 m
* Tokyo Bay, Japan, span 2250 m
* Chacao Channel, Chile, 2240 m
* Ares, Spain, 2198 m
* Ise Bay, Japan, 2100 m
* Kitan Strait, Japan 2100 m
* Coruna, Spain 2016 m.

Concerning cable-stayed bridges, the Stonecutter Bridge in Hong Kong, China, is expected to be the first one exceeding the 1000 m span limit (1018 m; to be built during 2003-2008) [5]. The Chongming Bridge in Shanghai, China, is planned to have even a longer span of 1200 m [9].

A unique cable-stayed floating bridge was proposed in the 1970's across the Gibraltar Strait, with 14 x 1000 m spans. Its deck would be of polyester laminate tube construction, accommodating 2 levels for road and 1 for rail traffic [10].

Also across the Gibraltar Strait, an enormous cable-stayed bridge was proposed in the 1980's with a main span of 8400 m. Even the side spans would be 3100 m and 4700 m in length and the towers 850 m and 1250 m in height, respectively. For such ultra long-span bridges, some new materials are provided [11].

11. EPILOGUE

Long-span bridges have interested us always, me and my wife, particularly great suspension bridges and also cable-stayed bridges (Tatara).

Hitherto, we have visited the following long-span cable-supported bridges (in order of main span), mentioned in Tables 1 and 2 : Akashi, Great Belt, Humber, Verrazano, Golden Gate, Höga Kusten, Fatih Sultan Mehmet, Bosporus, George Washington, Kurushima-3, Ponte 25 de Abril, Forth, Severn, Tatara, Transbay.

Excluding the 2 Istanbul suspension bridges, in other cases we were permitted to enter the top of the towers of the bridges mentioned above. The 3 Kojima-Sakaide suspension bridges in Table 1 (Minami, Kita, Shimotsui) we crossed by train and the Kurushima-2 Bridge on the Onomichi-Imabari Route we passed by ship, so those are not counted.

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TABLE 1: THE LEADING LONG-SPAN SUSPENSION BRIDGES WORLDWIDE
(completed, or under construction by 2002, span >700 m)

No.	Bridge	Span	Location	Year
1	Akashi-Kaikyo	1991 m	Kobe-Naruto, Japan	1998
2	Great Belt East	1624 m	Korsor, Denmark	1998
3	Runyang South	1490 m	Zhenjiang-Yangzhou, China	2005
4	Humber	1410 m	Kingston-upon-Hull, UK	1981
5	Jiangyin	1385 m	Jiangsu, China	1999
6	Tsing Ma	1377 m	Hong Kong, China	1997
7	Verrazano-Narrows	1298 m	New York, NY, USA	1964
8	Golden Gate	1280 m	San Francisco, CA, USA	1937
9	Höga Kusten	1210 m	Kramfors, Sweden	1997
10	Mackinac	1158 m	Mackinaw City, MI, USA	1957
11	Minami Bisan-seto	1100 m	Kojima-Sakaide, Japan	1988
12	Fatih Sultan Mehmet	1090 m	Istanbul, Turkey	1988
13	Bosporus	1074 m	Istanbul, Turkey	1973
14	George Washington	1067 m	New York, NY, USA	1931
15	Kurushima-3	1030 m	Onomichi-Imabari, Japan	1999
16	Kurushima-2	1020 m	Onomichi-Imabari, Japan	1999
17	Ponte 25 de Abril	1013 m	Lisbon, Portugal	1966
18	Forth Road	1006 m	Edinburgh, UK	1964
19	Kita Bisan-seto	990 m	Kojima-Sakaide, Japan	1988
20	Severn	988 m	Bristol, UK	1966
21	Yichang	960 m	Hubei, China	2001
22	Shimotsui-seto	940 m	Kojima-Sakaide, Japan	1988
23	Xiling	900 m	Sanxia, China	1996
24	Tiger Gate-1	888 m	Humen, China	1997
25	Ohnaruto	876 m	Kobe-Naruto, Japan	1985
26	Volgograd(1)	874 m	Volga River, Russia	1955
27	Tacoma(2)	853 m	Puget Sound, WA, USA	1950
28	Askoy	850 m	Bergen, Norway	1992
29	Innoshima	770 m	Onomichi-Imabari, Japan	1983
30	Akinada	750 m	Hiroshima, Japan	2000
31	Semipalatinsk	750 m	Irtysh River, Kazakhstan	2000
32	New Carquinez Strait	728 m	California, USA	2003
33	Hakucho	720 m	Muroran, Japan	1998
34	Angostura	712 m	Ciudad Bolivar, Venezuela	1967
35	Kanmon	712 m	Honshu-Kyushu, Japan	1973
36	Transbay	2 x 704 m	San Francisco, CA, USA	1936
37	Bronx-Whitestone	701 m	New York, NY, USA	1939
...	Kirjalansalmi	220 m	Parainen, Finland	1963
...	Sääksmäki	155 m	Valkeakoski, Finland	1963

(1)not standing
(2)first bridge collapsed in 1940

TABLE 2: THE LEADING LONG LONG-SPAN CABLE-STAYED BRIDGES

(completed, or under construction by 2002, span >471 m)

No.	Bridge	Span	Location	Year
1	Tatara	890 m	Onomichi-Imabari, Japan	1999
2	Pont de Normandie	856 m	Le Havre, France	1995
3	Nancha	628 m	Nanjing, China	2001
4	Wuhan Baishazhou	618 m	Wuhan, China	2000
5	Qingzhou Minjiang	605 m	Fuzhou, China	2001
6	Yangpu	602 m	Shanghai, China	1993
7	Xupu	590 m	Shanghai, China	1997
8	Meiko Central	590 m	Nagoya, Japan	1998
9	Rion-Antirion	3 x 560 m	Patras, Greece	2004
10	Skarnsundet	530 m	Trondheim Fjord, Norway	1991
11	Jueshi	518 m	Shantou, China	1998
12	Tsurumi Tsubasa	510 m	Yokohama, Japan	1994
1	Jingzhou	500 m	Hubei, China	2002
14	Ikuchi	490 m	Onomichi-Imabari, Japan	1991
15	Oresund	490 m	Denmark-Sweden	2000
16	Higashi-Kobe	485 m	Kobe, Japan	1992
17	Ehuang	480 m	Ezhou-Huangzhou, China	2002
18	Zhanjiang	480 m	Guangdong, China	2002
19	Ting Kau	475 m	Hong Kong, China	1998
20	Charleston	471 m	Cooper River, SC, USA	2006
...	Raippaluoto	250 m	Vaasa, Finland	1997
...	Kärkinen	240 m	Korpilahti, Finland	1997

TABLE 3: VARIOUS BRIDGE TYPES IN ORDER OF MAIN SPAN

(completed, or under construction by 2002, span >300 m)

No.	Type: Bridge	Span	Location	Year
1	Cable suspension: Akashi-Kaikyo	1991 m	Kobe-Naruto, Japan	1998
2	Cable-stayed: Tatara	890 m	Onomichi-Imabari, Japan	1999
3	Steel arch: Lupu	550 m	Shanghai, China	2003
4	Cantilever steel truss girder: Quebec Railway	549 m	Quebec, Canada	1917
5	Concrete arch: Wanxian	425 m	Sichuan, China	1997
6	Continuous steel truss girder: Ikitsuki	400 m	Nagasaki, Japan	1991
7	Double span swing: El Ferdan	340 m	Suez Canal, Egypt	2001
8	Chain suspension: Ponte Hercilio Luz	339 m	Florianopolis, Brazil	1926
9	Prestressed concrete girder: Stolmasundet	301 m	Austevoll, Norway	1998
10	Steel box girder: Ponte Costa e Silva	300 m	Rio de Janeiro, Brazil	1974

TABLE 4: SUCCESSIVE MAIN SPAN RECORDS DURING THE PAST TWO MILLENNIA

No.	Bridge (type or material)	Span	Location	Year
1	Martorell (stone arch)	37 m	Spain	B.C. 217
2	Trajan's Bridge(1) (timber)	52 m	Danube Riv., Romania	A.D. 104
3	Trezzo(1) (stone arch)	72 m	Italy	1377
4	Luding (chain suspension)	103 m	Dadu River, China	1705
5	Schaffhausen(1) (timber)	119 m(2)	Switzerland	1757
6	Schuylkill Falls(1) (suspension)	124 m	Philadelphia, PA, USA	1816
7	Union/Tweed (chain suspension)	137 m	Berwick, UK	1820
8	Menai Straits (chain suspension)	177 m	Anglesey, UK	1826
9	Fribourg(1) (suspension)	273 m	Switzerland	1834
10	Wheeling (suspension)	308 m	West Virginia, USA	1849
11	Lewiston(1) (suspension)	318 m(3)	Niagara River, USA-Canada	1851
12	Cincinnati (suspension)	322 m	Ohio River, USA	1867
13	Clifton(1) (suspension)	386 m	Niagara Falls, USA-Canada	1869
14	Brooklyn (suspension)	486 m	New York, NY, USA	1883
15	Firth of Forth (cantilever steel truss)	2 x 521 m	Edinburgh, UK	1890
16	Quebec (cantilever steel truss)	549 m	Quebec City, Canada	1917
17	Ambassador (suspension)	564 m	Detroit, MI, USA	1929
18	George Washington (suspension)	1067 m	New York, NY, USA	1931
19	Golden Gate (suspension)	1280 m	San Francisco, CA, USA	1937
20	Verrazano-Narrows (suspension)	1298 m	New York, NY, USA	1964
21	Humber Estuary (suspension)	1410 m	Hull, UK	1981
22	Akashi-Kaikyo (suspension)	1991 m	Kobe-Naruto, Japan	1998

(1) not standing
(2) 119 m actually consisted of 2 spans - middle support was temporarily removed after completion
(3) span 318 m c/c of towers - free span 257 m between river banks

AZHAR – YAMIN WHEAT PREDICTION MULTI-DIMENSIONAL MODEL 2003 : SCOPE AND APPLICATION

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ABSTRACT: *wheat is an important cereal and preferred grain for food in the Punjab province and country as a whole. Punjab Agriculture Department (PAD) every year sets wheat production targets at the outset of wheat crop season. These targets are then talked of in terms of surface water crises and every body fears the targets. The other factors such as supplementary irrigation through groundwater, conjunctive use of surface and groundwater, support price, fertilizer off take during Rabi or wheat crop period, temperature especially during the cold period at early crops growth stages, distribution and frequency of rainfall also affect the wheat production in bulk. So while fixing the production targets all these factors must be taken care of in addition to many other associated factors. To make effective predictions on wheat production, 10 years data sets (1992-93 to 2001-02) on the above enumerated crop production factors were manipulated to develop a concept of wheat prediction model, named in the paper as Azhar-Yamin Wheat Prediction Multidimensional Model-2003. The said model can be effectively used in making predictions at provincial level and can help policy makers to make import-export and support price decisions for this important food grain. The model can also lead to tentative quantification of the agricultural inputs which government may assure during a particular wheat season. It is recommended that Software Engineers should develop software of such models for effective and rational use both by the Punjab Agricultural Department (PAD) and policy makers.*

INTRODUCTION

Wheat *(Triticum aestivum)* is the most important world crop, which has played a significant role in the development of civilization. It is the preferred grain for food in both advanced and developing countries of the world (Martin *et al.*, 1976). It was sown in about 6080 thousand hectares in 2001-02 in Punjab with wheat production target set at 15.42 million tons (Directorate of Agri., 2002). The average yield of wheat crop in 2002-2003 targeted at 2791 kg per hectare and hence from a production area of 6003 thousand hectares wheat production expected is 15.64 million tons (Punjab Agri. Dept., 2002). The Punjab Agriculture Department sets targets of the crop each year. After setting targets, different speculations are linked with their achievement / accomplishment. There is no definite system or model to set such targets at provincial level, which can take good care of targets. While setting targets running simply after plain numerical values of average yield and production area without caring the socio-economical and agronomic crop production factors may be of no use (Javaid and Yamin, 2003). The targets must take care of crop production factors such as total canal water withdrawals, groundwater use, rainfall, temperature, fertilizer off take, support price in addition to production area under wheat. The production factors oriented data of few years may be coordinated to develop a mathematical model so as to fix crop production targets. Keeping aforementioned situation in view, the Azhar-Yamin Wheat Prediction Multidimensional Model 2003 a software concept has been developed with the following objectives:

1. To fix the tentative wheat production targets at provincial level

2. To predict the wheat production under a given set of crop production factors operative during a wheat season
3. To quantify the various agricultural inputs i.e fertilizer, production area, irrigation water etc required to hit the wheat production targets
4. To help policy makers to make support price and import-export decisions on wheat

APPROACH AND DATA COLLECTION

Different provincial and federal organizations/institutions consulted for data collection are tabulated below:

Sr. No	Organizations	Data category
1	Punjab Agriculture Department (PAD), Lahore	Area under wheat production
2	Directorate of Agriculture Crop Reporting Services, Lahore	Tubewell data
3	Punjab Irrigation & Power Department, Lahore	Canal withdrawals in Punjab
4	Fertilizer Cell, Directorate General of Agriculture (Extension), Lahore	Fertilizer off-take
5	a. Directorate of Land Reclamation, Punjab, Lahore b. Meteorological Dept. Jail Road, Lahore	Temperature & rainfall

The data values on the tabulated parameters were collected for the period from 1992-93 to 2001-02. These data oriented parameters were subject to multiple regression equation (Steel and Torrie, 1980) of the following type:

$$Y= a + b_1X_1 + b_2X_2 + b_3X_3 \text{ --------- } + b_nX_n$$

Where:

Y = dependent variable to be predicted
a = Y-intercept
b = Regression coefficient
X = independent variable on which prediction of dependent variable is based
n = any numerical number depending on size of the independent variable

The logarithmic values of the parameters were used to translate the multiple regression-based model into exponential format.

Model Description

The following different parameters affecting wheat production have been coordinated to design a model for predicting wheat production in Punjab:

1. Production area under wheat crop.
2. Total fertilizer use /off take during *Rabi* season.
3. Temperature fluctuations during crop growth period, especially length of the cold period favours the yield at early growth stage. A reduction in cold period

may affect the yields as compared to a little rise in temperature near or at maturity stage.

4. Rainfall: Normally it favours the crop growth and reduces evapotranspiration when coupled with low temperature.
5. Irrigation Water: Water shortage adversely affects the yield. The text of this dissertation reveals that over the last few years the wheat yield has increased besides canal water shortages. The detailed analyses reveal that deployment of tubewells in private sector has increased many fold compared to the non-water crises regime. The canal water deficit has been met with farmers' tubewell.
6. Support price incentive may also affect the wheat crop.

These parameters are grouped to develop a multiple regression-based model (Steel and Torrie, 1980). It entails multifacet crop production elements coordinated through multiple regression concepts, hence named as multidimensional model. The data of above enlisted different parameters mostly cover the data range from 1992-93 to 2001-02 (Table 1). This model is statistically justified with correlation coefficient value of (r=0.97**) at 0.020 standard error of estimate. The coefficient of determination (r^2) also has a weightage of 0.95**. The statistical functions reveal that concepts given in Azhar-Yamin Wheat Prediction Multidimensional Model 2003(A-Y Model) can be effectively employed to predict wheat production in Punjab. The statistical expression of A-Y Model is as under:

$$P = \frac{1.7783\times10^{-8}\left(a^{2.936}.\Sigma w^{0.015}.\gamma^{0.019}\right)}{X^{-0.234}.\,t^{-0.291}.\,f^{0.0236}}$$

Notation

P=Predicted wheat production in thousand tons

a=Area under wheat crop in thousand hectares

Σw= Summation of farmgate canal water availability during *Rabi* season and tubewell water availability in million acre foot (MAF).

γ= Total rainfall in millimeters from October to March

X= Support price in (rupees) per 40 kg wheat

t= Cumulative mean of minimum temperature from November to February in °C

f= Total nutrient off-take in Punjab during *Rabi* season from October to March in thousand nutrient tons.

Assumptions

It is stated that for the validity and application of the model all the parameters must have their data quantum at Punjab level. To work out the values at provincial level the following assumptions are made:

1. **For irrigation Water (Σw)**

i. Farmgate canal water availability is assumed 40% of the total canal withdrawal at head.

ii. (a) About 15.85AF water is pumped per one cusec tubewell during *Rabi* season to irrigate 12 acres of wheat of a farmer with 17 acres land holding (of which 5 acres may be supposed to undergo irrigation from canal on the principle that 30% of "Khatedars" land will receive canal irrigation during

Rabi); the wheat field is supposed to receive four irrigations of 3 acre inch each.

(b) Based on formula, Qt = dA (Q = discharge in ft^3/sec, t = time in hours, d = irrigation depth in inches, A = area in acres), it is assumed that one cusec tubewell discharge for 3 acre inch irrigation will take 3 hours to complete irrigation. However, to cover efficiency factor, it is finally assumed that it will take 4 hours to complete irrigation of one acre of land.

The factor 15.85AF/tubewell is worked out keeping in view that more than 80% of the farmers have land holding between 15-20 acres (Punjab Bureau of Statistics, 2000). However, 15+20/2=17 acres land holding is taken while calculating tubewells "running period" in discharge calculations.

(c) The number of total tubewells (public + private) multiplied by the factor 15.85 and divided by 10^6 gives the total discharge of tubewells in Punjab, supposed to be used for wheat crop in addition to canal water supplies (which irrigate 30% of farmers' land holding).

2. For Rainfall (γ)

Rainfall of eleven metrological stations representing Punjab received from each was summed up for each station and then a cumulative data value was obtained for 11 stations. The cumulative figure divided by number of stations gives an estimate of rainfall that occurs during a year.

3. Temperature (t)

The cumulative mean of minimum temperature will be worked out as under:

t = monthly mean of minimum temperature from Nov. to Feb. (four values) ÷ 4

4. Fertilizer (*f*)

It is assumed that the fertilizer Nutrient off take during *Rabi* season is used for wheat crop (the major *Rabi* crop). The fertilizer requirement for other *Rabi* crops is overlooked for the time being.

Sample Calculations

This model can also help estimate wheat production if a sound statistical database is available. It can be illustrated with the following numerical example

Example: database available

i. Area under wheat production 14664.99 thousand acres
ii. Canal water withdrawal at head during *Rabi* season= 18.892 x 10^6 AF
iii. Number of tubewells (Pvt + SCARP) working during *Rabi* season= 500631
iv. Support price of wheat Rs = 300 / 40 kg
v. a) Nitrogen fertilizer nutrient off take = 700 x 10^3 tons
   b) Phosphorus fertilizer nutrient off take during *Rabi* = 300x10^3 tons
   c) Potassic fertilizer nutrient off take during *Rabi* = 100x10^3
vi. Rainfall (from Oct. to March) = 114 mm, Mean minimum temperature from Nov. to Feb. = 8.24 °C
vii. P (production) (thousand tons) =?

Solution:
Azhar-Yamin Multidimensional Wheat Prediction Model-2003

$$P = \frac{1.7783\times10^{-8}\left(a^{2.936}.\sum w^{0.015}.\gamma^{0.019}\right)}{X^{-0.234}.\,t^{-0.291}.\,f^{0.0236}}$$

The values of "a" and "X" (area and support price) are given. The value of "a" as per model requirement should be in thousand hectares i.e.

i. **a** = 14664.99 thousand acres = 14664.99/2.4711 =5934.6 thousand hectare
ii. **Canal withdrawal** = 18.892×10^{6} AF = 18.892 MAF at farm gate with 60% losses = 7.55 MAF
iii. **Number of tubewells** = 500631. Total discharge @ 15.85 AF/tubewell during *Rabi* season = 7935001.4 AF = 7.93 MAF, $\sum w$ =(ii)+(iii) = 7.55+7.93 =15.48 MAF
iv. **X** (Support price) = Rs 300 /40 kg wheat
v. *f*= Total fertilizer nutrient off take during *Rabi* (a+b+c) = 1100,000 tons = 1100 thousand nutrient tons
vi. γ= 114 mm
vii. **t** = 8.24 ^{0}C

Substituting values in Azhar – Yamin Multidimensional Wheat Prediction Model-2003

$$P = \frac{1.7783 \times 10^{-8}\,(5934.6)^{2.936}.\;(15.48)^{0.015}.\,(114)^{0.019}}{(300)^{-0.234}.(8.24)^{-0.291}.(1100)^{0.0236}}$$

$$= \frac{2430.0213}{0.1681108} = 14455 \text{ Thousand tons}$$

So wheat production in Punjab under the given data can be predicted as 14455 thousand tons

SCOPE AND APPLICATION

The A.Y Prediction Model-2003 with its specifications can be used to estimate wheat production in Punjab under any given set of wheat production economical, physical, chemical and climatic conditions. Besides making prediction of wheat production at provincial level and helping decision on wheat export-import matters, the A.Y Model can give a tentative targets of inputs which government has to arrange during wheat season in a particular year. These parameters are highly correlated in the model (r=0.97**). It is evident from sub-fig of Fig.1 that deviation of the A-Y wheat prediction model values from the observed wheat production values in Punjab varies from ±0.95 to 2.38% for the actual scenarios of different years. This difference is statistically justified and is admissible at 5 percent probability level under field conditions. In Fig.1 we cannot use the years in place of scenario(s), because each scenario itself accommodates the data of rainfall, temperature, farm gate water availability etc. It reflects the accuracy with which the A-Y wheat prediction multidimensional model-2003 can be used to make predictions.

Modified Version of A-Y Wheat Prediction Multidimensional Model-2003 for making Predictions

For making predictions we have to work out some parameters of the Azhar-Yamin Multidimensional Wheat Prediction Model 2003. The following modified versions of the said model can be followed for quantification of a variable to predict the degree of achievement of any production target i.e.

a. The fertilizer off take desired during *Rabi* season and which need to be arranged can be calculated as per following formula

$$f^{0.0236} = \frac{1.7783\text{x}10^{-8}(a^{2.936}.\Sigma \mathbf{w}^{0.015}.\gamma^{0.019})}{X^{-0.234}.t^{-0.291}.P}$$

It can be solved either through logarithmic function or by exponential function:

i. **By logarithmic function**

Taking log of both sides of the formula

$$\log(f)^{0.0236} = \log\left(\frac{1.7783\text{x}10^{-8}(a^{2.936}.\Sigma \mathbf{w}^{0.015}.\gamma^{0.019})}{X^{-0.234}.t^{-0.291}.P}\right)$$

$$0.023\log(f) = \log\left(\frac{1.7783\text{x}10^{-8}(a^{2.936}.\Sigma \mathbf{w}^{0.015}.\gamma^{0.019})}{X^{-0.234}.t^{-0.291}.P}\right)$$

$$\log(f) = \log\left(\frac{1.7783\text{x}10^{-8}(a^{2.936}.\Sigma \mathbf{w}^{0.015}.\gamma^{0.019})}{X^{-0.234}.t^{-0.291}.P}\right)/0.023$$

$$f = \text{Antilog}\left[\log\left(\frac{1.7783\text{x}10^{-8}(a^{2.936}.\Sigma \mathbf{w}^{0.015}.\gamma^{0.019})}{X^{-0.234}.t^{-0.291}.P}\right)/0.023\right]$$

ii. **By exponential function**

$$f^{0.0236} = \frac{1.7783\text{x}10^{-8}(a^{2.936}.\Sigma \mathbf{w}^{0.015}.\gamma^{0.019})}{X^{-0.234}.t^{-0.291}.P}$$

$$f = \sqrt[0.0236]{\frac{1.7783\text{x}10^{-8}(a^{2.936}.\Sigma \mathbf{w}^{0.015}.\gamma^{0.019})}{X^{-0.234}.t^{-0.291}.P}}$$

b. The area under wheat production can also be worked out by adoption of exponential function i.e.

$$a^{2.936} = \frac{P.X^{-0.234}.t^{-0.291}.f^{0.0236}}{1.7783\text{x}10^{-8}.\Sigma \mathbf{w}^{0.015}.\gamma^{0.019}}$$

$$a = \sqrt[2.936]{\frac{P \cdot X^{-0.234} \cdot t^{-0.291} \cdot f^{0.0236}}{1.7783x10^{-8} \cdot \Sigma w^{0.015} \cdot \gamma^{0.019}}}$$

c. Total water (Σw) i.e. summation of canal water farmgate availability and groundwater during *Rabi* can be computed as under:

$$\Sigma w^{0.015} = \frac{P \cdot X^{-0.234} \cdot t^{-0.291} \cdot f^{0.0236}}{1.7783x10^{-8} \cdot a^{2.936} \cdot \gamma^{0.019}}$$

$$\Sigma w = \sqrt[0.015]{\frac{P \cdot X^{-0.234} \cdot t^{-0.291} \cdot f^{0.0236}}{1.7783x10^{-8} \cdot a^{2.936} \cdot \gamma^{0.019}}}$$

d. Guidelines on support price can be obtained as per following form of model.

$$X^{-0.234} = \frac{1.7783x10^{-8}(a^{2.936} \cdot \Sigma w^{0.015} \cdot \gamma^{0.019})}{t^{-0.291} \cdot f^{0.0236} \cdot P}$$

$$X = \sqrt[-0.234]{\frac{1.7783x10^{-8}(a^{2.936} \cdot \Sigma w^{0.015} \cdot \gamma^{0.019})}{t^{-0.291} \cdot f^{0.0236} \cdot P}}$$

Predicting Wheat Production for 2002-2003

The Punjab Agri. Dept (PAD) has set the target of production area at 6180 thousand hectares for the year 2002-03. Assuming that canal withdrawals, number of tubewells and fertilizer off take remain same as in the previous year, the wheat production from this target area can be predicted by using the Scenario S10 (Table 1) except production area with the help of the A-Y wheat prediction multidimensional model 2003.

$$P = \frac{1.7783 \times 10^{-8} [6180]^{2.936} [14.04]^{0.015} [46.86]^{0.019}}{[300]^{-0.234} [7.67]^{-0.291} [994.62]^{0.0236}}$$

$$P = \frac{1.7783 \times 10^{-8} (1.5111x10^{11})}{0.1712469} = \frac{2561.367}{0.1712469}$$

= 14957.16 thousand tons

This reflects that if no natural calamity occurs and the previous year's conditions persist, the desired target of 15640 thousand tons for 2002-03 cannot be achieved. There will be a deficit of 642.84 thousand tons. To accomplish this target, we have to concentrate on efficient use of nutrients and adequate water supply. The rainfall if on time especially during February- March may overcome the water shortage and cover the deficit. In such a case the substitution of rainfall value in the formula may predict the increase in yield at Punjab level. In regime of canal water shortage, the rainfall only may give us a bumper wheat crop, may be 12 to 18% increase from the last year. If there is no rainfall and water scarcity prevails, then wheat production target may be affected. Anyhow, the rainfall quantity put in the model can reflect the near exact situation.

Various options of the models can be manipulated to quantify the production factors i.e.

i. adjusting $\sum w$
ii. adjusting f of the model under discussion

Let us manipulate the model to fix some standards for the achievement of the wheat production target in 2002-2003.

- The support price (x) and area under production are directly related to each other. The timely decision on support price may act as an incentive to the farmers for bringing more area under wheat cultivation. If other conditions are kept constant, then before the wheat season farmers must be convinced to achieve the target of production area under wheat and it should be assured that target is timely achieved at the end of the season. The target area should not be simply calculated on the basis of ± average yield / area of wheat for a given yield production target, rather other factors responsible to affect yield production should also be taken care of while fixing a target area for wheat production.

These factors are taken care of in the Azhar-Yamin Multidimensional Wheat Prediction Model-2003. The target area fixed by the PAD to achieve the yield production target of 15640 thousand tons (keeping previous years conditions in view) can be different from what has been fixed. Let us see how much it should have been fixed:

$$a^{2.936} = \frac{15640 \times (300)^{-0.234} \times (7.67)^{-0.291} \times (994.62)^{0.0236}}{1.7783\times10^{-8} \times (14.04)^{0.015} \times (46.86)^{0.019}}$$

=6173.11 thousand hectares

So, it should have been fixed at 6173.11 instead of 6180 thousand hectares to achieve the desired target of 15640 thousand tons. There is a difference of about seven thousand tons.

If the area is the same as fixed by the PAD and other conditions more or less act as the same as those in the previous year, then support price of wheat may activate the farming community to a good crop husbandry.

The support price that if has to be announced, should be Rs. 296/40kg of wheat as per Azhar-Yamin Wheat Prediction Multidimensional Model 2003. Therefore, there is no need to enhance the existing support price of wheat i.e. Rs 300/40 kg of wheat at least for the current *Rabi* season However, announcement of support price next year well in advance to cropping may lead to good crop production in 2003-04.

$$x^{-0.234} = \frac{1.7783\times10^{-8}(a^{2.936} . \sum w^{0.015} . \gamma^{0.019})}{t^{-0.291} . f^{0.0236} . P}$$

$$X = \sqrt[-0.234]{\frac{1.7783\times10^{-8}(6180)^{2.936} . (\Sigma 14.04)^{0.015} . (46.86)^{0.019}}{(7.67)^{-0.291} . (994.62)^{0.0236} . (15640)}}$$

296 ≃ 300

Now, when the crop is sown and neither the support price nor the production area has been increased, then we are left with two options i.e.

i. Assurance of Fertilizer off take during *Rabi*
ii. Assurance of Irrigation water requirement regulation (However, this decision may be affected by natural rainfall)

- Crop fertilization accompanied by irrigation water assurance can also help achieve the yield production target. Fertilizer can increase yield upto 50% and can help minimize crop production gap. The inadequate and imbalanced use of fertilizers contributes towards 30-80% yield gap (Ahmad, 1998). Timely availability of fertilizers at doorstep of the farmers at the time of crop fertilization is conducive to achieve the target. The PAD should keep an eye through its extension staff to assure fertilizer distribution during the *Rabi* season. Total fertilizer off-take that should be assured may be worked out as per model's calculations i.e.

$$f^{0.0236} = \frac{1.7783\times10^{-8}(a^{2.936} . \Sigma w^{0.015} . Y^{0.019})}{X^{-0.234} . t^{-0.291} . P}$$

$$0.0236 \text{Log} f = \text{Log}\left(\frac{1.7783\times10^{-8}(6180)^{2.936} . (14.04)^{0.015} . (46.86)^{0.019}}{(300)^{-0.234} (7.67)^{-0.291} . 15640}\right)$$

$\log f = \log (1.1808317)/0.0236$

$\log f = 0.072188/0.0236=3.0588137$

f = Antilog 3.0588137 = 1145 thousand tons

Therefore, contrary to production area and support price, the fertilizer off take has to be targeted at 1145 thousand nutrient tons against the previous year's off take of about 995 thousand tons.

- Water is lifeblood of agriculture. The Punjab province is running short of surface water (canal) supplies. The canal water allocated to Punjab during *Rabi* season as per Water Apportionment Accord -1991 is still insufficient at farm gate as per requirement of the crop. The farmers are exploiting groundwater even for this low delta crop. The data reveal that besides low canal supplies the yield increased at the outset of water crises regime in 1999-2000 (which subsequently began to decline due to some reasons). The farmers turned positive this negative relationship between shrinking canal water supplies and expanding wheat yield through ground water exploitation.

The data on Σw (Table 1) reveal that total farm gate availability increased with the increasing canal water crises; obviously, it was the contribution of groundwater. The

$\sum w$ in this season after sowing crop must be strictly observed and scientifically managed. The $\sum w$ in the light of Azhar-Yamin MDWP Model-2003 to be assured may be computed as under (keeping other conditions same, as has prevailed during *Rabi* 2001-02):

$$\sum \mathbf{w}^{0.015} = \frac{P \,.\, X^{-0.234} \,.\, t^{-0.291} \,.\, f^{0.0236}}{1.7783\times10^{-8} \,.\, a^{2.936} \,.\, \gamma^{0.019}}$$

$0.015 \log \sum w = \log 1.0369724$
$0.015 \log \sum w = 0.0157671$
$\log \sum w = 0.0157671/0.015$
$\sum w =$ Antilog $1.0511456 = 11.249$
$= 11.25$

The canal withdrawal during *Rabi* 2001-02 was 9.779 MAF, which at 60% losses amounted to3.91 MAF. The remaining contribution was expected from groundwater during *Rabi* for wheat crop only. The total farm gate water from canal + tubewells was estimated to 14.04 MAF. This year prices of diesel oil, deepening water table and other factors reveal that farmers will not pump water to the extent followed last year. The canal water shortage again is noted to the extent of 35% during early December 2002. This may affect the crop target reported in the press.

The A-Y Wheat Prediction Multidimensional Model 2003 computations reveal that to achieve the target of wheat production (with the assumptions of last years climatic and input conditions) fixed by Punjab Agriculture Department during 2002-03, the production elements need to be quantified as under:

$\sum$**w** (Farm gate irrigation water, inclusive of groundwater in *Rabi*)=11.25 MAF
or
a (area under wheat production)= 6173.11 thousand ha
or
f (Total nutrient off take during *Rabi*) = 1145 thousand nutrient tons
or
X (support price) =296 Rs /40 kg wheat

Thus, area (6180 thousand ha) and support price of Rs. 300/ 40 kg wheat during 2002-03 is almost justified by the model. However, fertilizer off take as per models calculations must be higher. This is the situation if no rainfall occurs. The yield may increase in case of sufficient and timely rainfall blesses our wheat crop.

CONCLUSIONS & RECOMMENDATIONS

1. Statistically justified A.Y. Model can be effectively used to predict wheat production in Punjab

2. The model can help to make decisions on export-import matters of wheat

3. Model can help to predict the input targets, which Government may arrange during a particular wheat year.

4. It is recommended that PAD should define some sound criteria to fix the targets of wheat production in Punjab. The target should not be simply based on qualitative effects of inputs, shrinking or expanding. These targets must base on quantitative effects of inputs and other elements directly or indirectly relating to crop production. Model like that used in this text should be worked out; software developed and updated for risk analysis and mitigation measures.

5. It is also pointed out that while fixing yield production targets the population growth rate in addition to other factors must be taken care of. It is also worth mentioning that land and water resources if not properly managed, we have to retreat the crop production targets every year. This will not only depress our recent hope of exporting wheat but it would also yield to food sustainability concern in view of ever increasing population growth of Punjab. It is therefore, imperative to broaden the land and water resource base in addition to managing to the best potential of the existing recourses.

6. A software of statistically justified simple multiple regression based A-Y Model 2003 should be developed by the Software Engineers.

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Table 1: Database for development of A-Y Multidimensional Wheat Prediction Model 2003

Scenario	Year	Production Area (000'ha)	Total tubewells (pvt+SCARP)	Water* Requirement at farmgate (MAF)	Minimum Temp(°C) Oct.-Nov.	Canal withdrawal at head(*Rabi*) (MAF)	Canal water at Farmgate @ 60% losses (MAF)	*Rabi* Rainfall (mm) Oct-March	Deficit (MAF)	Tubewell+ water approximation (MAF)	Total water at farmgate (C+T) ∑w
S1	1992-93	5960.5	328261	14.73	7.71	20.859	8.34	170.90	6.39	5.20	13.54
S2	1993-94	5770.7	396459	14.26	7.78	17.513	7.00	49.26	7.26	6.28	13.28
S3	1994-95	5902.3	415271	14.59	8.30	20.296	8.11	138.06	6.48	5.58	14.69
S4	1995-96	5973.5	435228	14.76	7.84	20.232	8.09	158.28	6.67	6.89	14.98
S5	1996-97	5839.9	452431	14.43	6.76	19.246	7.70	84.30	6.73	7.17	14.87
S6	1997-98	5934.6	473667	14.66	7.8	17.985	7.19	210.90	7.47	7.50	14.69
S7	1998-99	5934.6	500631	14.66	8.24	18.892	7.56	114.95	7.10	7.93	15.49
S8	1999-2000	6180.3	543243	15.27	7.65	16.418	6.57	64.79	8.70	8.60	15.17
S9	2000-01	6244	589473	15.42	7.37	11.354	4.54	39.70	10.88	9.34	13.88
S10	2001-02	6080	639637	15.02	7.67	9.779	3.91	46.86	11.11	10.13	14.04

*It is assumed that wheat requires 4 irrigations each with average 3 inch irrigation (1AF), + It is assumed that more than 80% farmers have 15-20 acres of land. On the average if land holding is 17 acres and 30% of it is canal irrigated (say 5 acres) then rest (12 acres) is supposed to be irrigated with tubewells. To irrigate 12 acres @ 3 acres inch 4 irrigations each of 3 acre inch, the time assumed to irrigate an acre by a tubewell of one cusec is 3 hours. Thus, a tubewell is assumed to run for 8 days for *Rabi* season for pumping 15.85 AF water. +Calculated from the figure of 1999-2000 @ 51% growth rate is for the year of 1995-96 to 1999-2000.

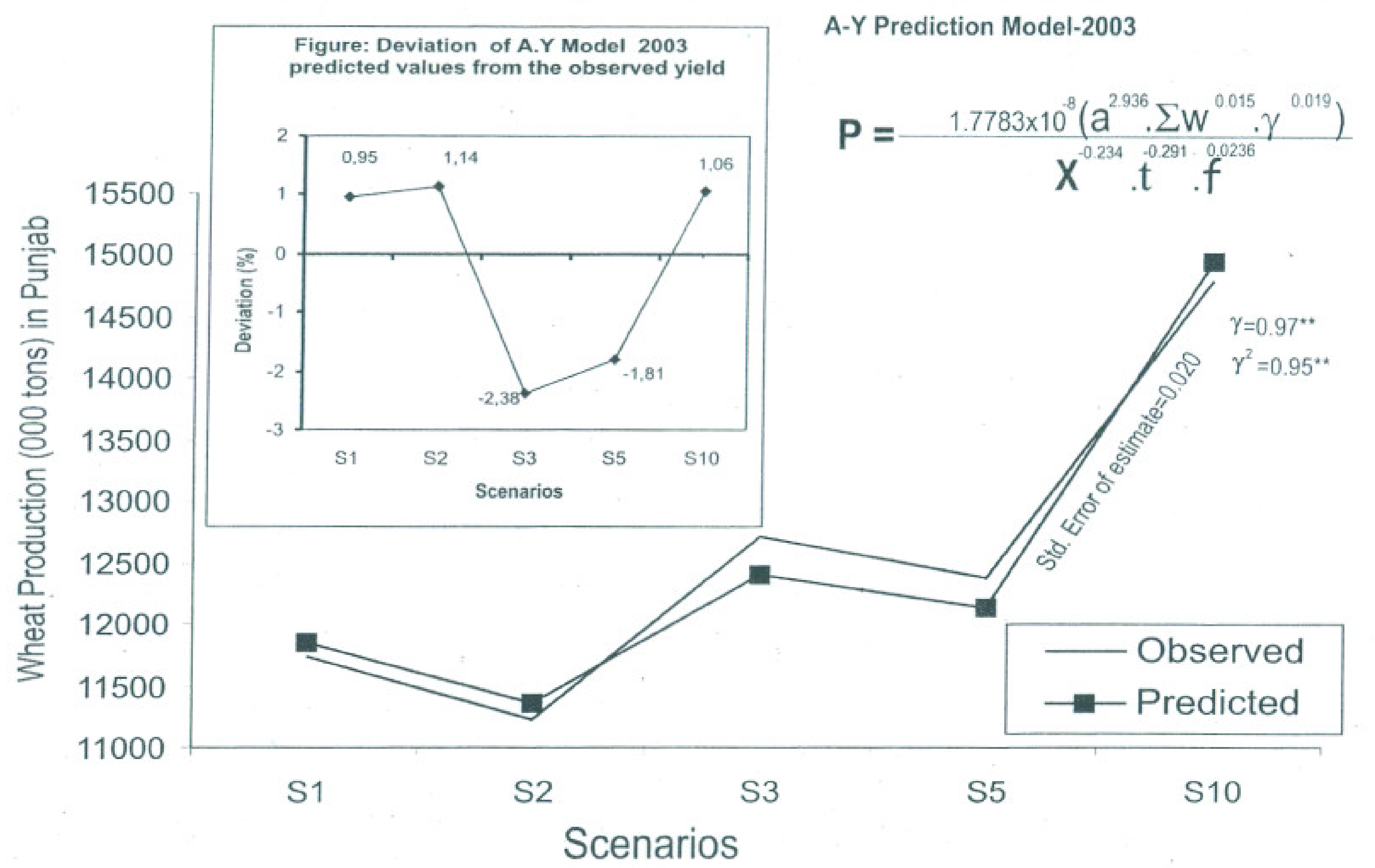

Figure 1: The observed and A. Y Model 2003 predicted wheat production reliability test under different Scenarios

MATHEMATICAL MODELLING OF ALLUVIAL RIVERS REALITY AND MYTH. (PART 2 : SPECIAL ISSUES)

By

Z. Cao and P. A. Carling

The last half a century has seen more and more developments and applications of mathematical models for fluvial flow, sediment transport and morphological evolution. However, the quality of this modelling practice has emerged as a crucial issue for concern, which is widely viewed as the key that could unlock the full potential of computational fluvial hydraulics. The major factors affecting the modelling quality comprise : (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 models veracity. An overview of mathematical models for alluvial rivers is provided in this and the companion paper "Part 1 : General review". This paper is the second part, dealing with three special issues of mathematical river models. First, turbulence closure models are highlighted, particularly with respect to the role of sediment in modulating turbulence and its implications for adapting turbulence closure models for fluvial sediment-laden flows. Second, the bottom boundary conditions are discussed in detail as one of the main sources of model uncertainty. And third, the commonly used calibration and verification/validation methodology in mathematical river modelling is addressed. It is argued that model calibration can be subjective, verification is impossible because models are not closed systems, and validation does not necessarily establish model truth. Confirmation of observations by models only supports model probability, rather than demonstrating model veracity. It is vital for model developers and end-users to keep aware of what mathematical river models can realistically reflect, and therefore avoid misleading decision-making. Additionally, some strategies are proposed which can improve the practice of mathematical river modelling.

NOTATION

C	local volumetric sediment concentration
C_a	near-bed volumetric sediment concentration
C_{ac}	near-bed equalibrium volumetric sediment concentration
D	sediment deposition flux
E	sediment entrainment flux
h	flow depth
Rd	ratio of sediment particle size to turbulence micro-scale
St	sediment particle Stokes number
U, u	velocities
w	effective sediment fall velocity
w_0	single sediment particle fall velocity
x	streamwise coordinate

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z, z_1 Rouse number

z/h relative distance above bed

ν_t, ν_{tc} turbulent eddy viscosities of sediment-laden and clear-water flows respectively

1. INTRODUCTION

Fluvial sediment transport poses great challenges for river scientists and engineers. The essence of the discipline is the interaction between the fluid (water) and the solid (dispersed sediment particles) phases. The exposure of the fluvial systems to the natural and variable environment (climatic, geological, ecological and social, etc.) adds to the complexity of the process of sediment transport and the resulting morphological evolution of rivers. The earlier efforts in mathematical river modelling were almost exclusively built on traditional fluvial hydraulics – that is, one-dimensional (1D) and two-dimensional (2D) Saint-Venant equations. The 1D and 2D models are at present widely used in engineering practice ; yet the future of mathematical river modelling will undoubtedly be the more advanced full 3D computational fluid dynamics.

As stated in the companion paper, the quality of 3D modelling is strongly grounded on the turbulence closure model and bottom boundary conditions that must be implemented. The issues in this connection have been underappreciated in the current river modelling practice. Essentially the turbulence closure models in current 3D river models are the same as those developed for single-phase flows, without incorporating the impacts of sediment carried by the flow. The bottom boundary conditions, prescribed in existing 3D (and vertical 2D) models, are rarely established with adequate recognition of the underlying assumptions, and bear considerable ambiguity. An example of improper formulation of bottom boundary conditions can be found in De Cesare's paper for reservoir sedimentation induced by turbidity currents, and also discussed by Cao.

Further, mathematical river models are never closed systems, for which substantial empiricism is indispensable for the closure of the governing equations and boundary conditions. However, verification on a rigorous basis is possible only for well-established closed systems, which makes the calibration and verification/validation methodology widely used in mathematical river modelling problematic.

As the second part of the present overview of mathematical river modelling, this paper is concerned with three special issues. Specifically, the turbulence closure model is discussed with respect to sediment effects, the bottom boundary conditions are highlighted, and the commonly used terminology calibration and verification/validation as assertions of model veracity are addressed. Additionally, within the current knowledge framework of fluvial sediment transport, suggestions are made to improve the practice of mathematical river modelling.

2. TURBULENCE CLOSURE FOR FLUVIAL SEDIMENT-LADEN FLOWS

For single-phase flows the current state of the art for turbulence closure models can be found in recent literature. Often the more complicated closure models (e.g. Reynolds stress model) can be expected to give better resolution of the turbulent flow structure than simpler closures. However, the advantage may be marginal or limited if only the mean flow quantities are of major interest. Also the significantly higher computational cost of the complete Reynolds stress closure models can make them less attractive for large-scale river problems ; a viable balance between the complexity and capacity of turbulence closure models is thus essential. Likely the $\kappa - \varepsilon$ two-equation model is a good choice.

Although turbulence closure models have seen successful applications in a wide range of engineering areas. It is essential to recognise the particular prominence of turbulence closure as a major source of errors of numerical river models. Numerical models for river flows (even without sediment), while offering increasing predictive power and potential, are not yet sufficiently well-established to be applied routinely to complex 3D fluvial flows, unless only a rough qualitative knowledge is to be sought. This is determined by the complicated channel topography, boundaries and composition of bed materials, in sharp contrast to flow problems with adequately clearly resolved boundaries seen in other industrial areas such as aircraft, automotive, heating and ventilation design, etc.

The behaviour of turbulence interaction with complicated boundaries (e.g. gravel or dune-covered riverbeds) remains poorly understood and formulated. It must not be forgotten that considerable expertise, physical insight and experience are vital for meaningful numerical solutions to be acquired and for the limitations of model results to be properly appreciated and interpreted. In the following section particular attention is paid to the role of sediment in modulating turbulence and to its implications for adapting clear-water turbulence closure models to the modelling of fluvial sediment-laden flows.

2.1. The role of sediment in modulating turbulence and its implication

Sediments are known to alter the structure of the turbulent flow, by which they are carried, which in turn affects the transport of sediments. The sediment effects can be appreciable as the volumetric sediment concentration reaches about the order of magnitude of 10^{-6}, which is quite common in natural rivers. The interaction between sediment and turbulent flow is of fundamental interest in the mechanics of sediment transport specifically and also revelant in the field of two-phase flows in general.

As with the issue of flow resistance influenced by sediment, there has been considerable controversy over the behaviour of turbulence modulation (enhancement or attenuation) by sediment, along with its implication for turbulence closure models for such flows. This topic has been the theme of a large number of experimental studies. Elata and Ippen's measurements of streamwise velocity fluctuations by means of an impact-tube transducer for nearly neutrally buoyant particle-laden flows with high particle concentrations show increased turbulence intensity. Muller suggests increased turbulence intensity based on LDV (Laser Doppler Velocimeter) measured data for coarse sand-laden flows. Van Ingen reports that no appreciable change in turbulence intensity occurs due to fine sands. Wang and Qian's experiments for both natural sands and neutrally buoyant plastic particles using LDV indicate appreciable decrease in turbulence intensity. Lyn's results using LDV for both equilibrium and starved-bed situations with natural sands reveal that the effects of suspended sediments on turbulent characteristics are, at best, moderate or negligible. Wang and Larsen propose that turbulence may be intensified in response to the addition of coarse particles as bed-load. The recent experiments of Best using phase Doppler anemometry show both enhancement and attenuation of turbulence intensity, depending on the distance from the bed with differing values of particle Stokes number *St* and the ratio *Rd* of particle size-to-turbulence length scale. Muste and Patel used discriminator LDV to measure the separate velocities of the suspended sand and underlying water in dilute sediment-laden flows. Their results show that the sreamwise turbulence intensities of water and sand are practically the same, whereas the vertical sediment turbulence intensities are dampened throughout the flow depth, which are more pronounced near the bed. The existence of the slip velocity between the fluid and particle phases has been reported by others. This latter concept has been used to improve the calculation of suspended load. The measurements by Cellino and Graf using a non-intrusive sonar instrument indicate that turbulence is suppressed by suspended sediment. The studies listed immediately above are representative and by no means exhaustive.

It is noteworthy to mention the findings concerning the particle-air two-phase flows that share considerable similarity to suspended sediment-laden flows. It has been found that turbulence may be enhanced when the suspended particles are larger than the turbulence length scale, or suppressed when they are so fine as to be enclosed within turbulent eddies. In particular, it has been shown that turbulence is attenuated or enhanced by suspended particles respectively in relation to small or large values of *St* and *Rd*. The critical values of *St* and *Rd* for transition between turbulence attenuation and enhancement are about 1· 0 and 0· 1 respectively. Fig. 1 summarises the effects of particle size on turbulence. Supporting the above finding, Nino and Garcia claim that turbulence may be attenuated or enhanced by suspended sediment depending on sediment particle size.

For the purpose of applied mathematical river modelling, the role of sediment in modulating turbulence must be properly formulated and incorporated into the closure modules for sediment-laden flows. Attempts have been made to modify the Prandtl's mixing length concept to represent the idealised steady and uniform suspended sediment-laden flow. But the modifications are based on either simple and poor reasoning or upon analytical results that exhibit great scatter. For more complicated flow situations, the most popular turbulence closure models are essentially those initially developed for single-phase flows in widely differing forms. The effects of suspended sediment on the flow are in fact not fully taken into account. At best, they are partly considered only by the buoyancy production term in the momentum conservation equation or the modified turbulent eddy viscosity involving a stratification parameter (the gradient Richardson number or alternatively the Monin-Obukhov length scale). The current state of the art with regard to this aspect is described by ASCE. In spite of the great efforts in respect of numerical modelling of sediment-carrying flows, there has been little essential progress, if any, in the turbulence closure models relevant to such flows. This status, in the sense of applications, evidently does not parallel the endeavour to enhance the understanding of the mechanism of turbulence modulation by sediments. The substantial variability of parameters in stratification analogy-based models appears to flag the existence of other mechanisms, in addition to stratification, that are responsible for turbulence modification. One may contend that formulating the recent finding regarding the general particle-fluid two-phase flows (as illustrated in Fig. 1) can be the way forward. It is, at least currently, an encouraging standpoint with which to resolve the controversy over the nature (attenuation or enhancement) of turbulence modulation by particles.

2.2. Turbulent diffusivity of suspended sediment

One essential aspect of the widely used algebraic slip mixture (ASM) models for suspended sediment is the closure of turbulent diffusivity (ε_s) for suspended sediment.

Conventionally this is determined using a simple analogy between turbulent mass transfer and Fickian diffusion, and a rather crude expression relating it to turbulent eddy viscosity – that is :

$$(1) \qquad \varepsilon_s = \beta \nu_t$$

The proportional factor β in this expression is often related to the Schmidt number. It is well known that specifying β is purely empirical. Recall the simplest steady and uniform flow situations mentioned earlier – there is little confidence in specifying its value. It is this factor that make it necessary to tune the Rouse parameter (actually β or the Schmidt number) to reconcile the vertical sediment concentration profiles to measured data. Recent laboratory studies of β can be found in Cellio and Graf. Yet the most comprehensive analysis of a large measured dataset, little known to the Western community, is due to Xie and Zou. Fig. 2 shows the result of the analysis, which indicates a rather scattered relationship between the primary Rouse parameter z ($z = w_0/\kappa u_*$ where w_0 is the settling velocity of a single sediment particle, κ is the Von Karman constant and u_* is the shear velocity) and its counterpart z_1 ($z_1 = w_0/\beta\,\kappa\, u_*$) calibrated from measured dataset. It can be used as the first tool to approximate the proportional factor β, given the value of the primary Rouse parameter. Importantly, it is necessary to note that the β-value can be either less or larger than unity. Thus the stratification analogy-based model is obviously problematic as it always gives $\beta < 1$. The authors' experience shows that the role of turbulent diffusivity of suspended sediment in mathematical river modelling is never trivial. Unfortunately this is rarely addressed in mathematical models, leaving considerable ambiguity, especially for strongly 3D flows (e.g. in compound channels).

3. THE BOTTOM BOUNDARY CONDITIONS

3.1. Net flux of sediment exchange versus bottom boundary condition

Generic to any spatially dimensional mathematical river models, formulating the net flux of sediment exchange with bed material is of fundamental importance for fluvial sediment transport. Generally speaking, two distinct mechanisms for the exchange process are involved. On the one hand, sediment particles on the bed surface may be entrained by turbulent fluid motions. On the other hand, suspended sediment particles tend to deposit under gravitational action. The mean net flux of sediment exchange vanishes only under equilibrium conditions, while in the general non-equilibrium cases there is a finite net flux that results in the variation of sediment transport rate and bed deformation (or morphological changes). In 3D and vertical 2D models, the flux is manifested in the bottom boundary condition for sediment phase – that is :

$$(2) \qquad \left(\omega C + \varepsilon_s \frac{\partial C}{\partial C}\right)_{z=a} = D - E$$

Where C is mean volumetric sediment concentration ; D and E are respectively sediment deposition and entrainment fluxes across the computational near-bed boundary defined at $z = a$ above the bed, with z being the vertical coordinate ; and ω is the effective setting velocity of sediment. Although in depth-averaged 2D and 1D models, it is represented in the source term of sediment continuity equation, its significance in mathematical river modelling is also essential.

There is little dispute that the deposition flux can be calculated practically using the local near-bed sediment concentration C_a and ω by $D = \omega C_a$. Also ω can be approximated by the setting velocity ω_0 of a single particle and local sediment concentration with a validated relationship. Thus specifying bed sediment entrainment flux is the key to determination of the net flux of exchange. It follows from equation (2) that :

$$(3) \qquad -\left(\varepsilon_s \frac{\partial C}{\partial z}\right)_{z=a} = E$$

There appears to be plethora of empirical functions for bed sediment entrainment. However none of these yields generally satisfactory results for various particle sizes. The popular yet traditional approach to specifying bed sediment entrainment flux is based on the assumption that entrainment occurs always at the same rate as it does under equilibrium conditions. This has been used by other researchers including many Chinese researchers. As in equilibrium conditions, the entrainment flux is equal to the deposition flux, therefore the bed sediment entrainment flux can be computed using equilibrium near-bed sediment concentration C_{ae} and effective setting velocity. Therefore one obtains from equation (3) that :

$$(4) \qquad -\left(\varepsilon_s \frac{\partial C}{\partial z}\right)_{z=a} = E = \omega C_{ae}$$

Unhappily the equilibrium near-bed concentration has had to be determined either directly from empirical relationships, or indirectly from suspended sediment transport capacity formulae with the aid of presumed velocity and concentration profiles over the flow depth. The equilibrium near-bed sediment concentration has mostly been approximated using one of a range of empirical relationships. These relationships may deviate from each other greatly and give unrealistically large concentrations. So far as suspended sediment transport capacity is concerned, previous relationships usually exhibit large scatter with respect to the parameters involved. Further, the velocity and concentration profiles can come with substantial uncertainty even for idealized steady and uniform channel flows as mentioned earlier.

Being widely used and even explicitly embodied in the governing equations of mathematical models, the above-stated approach to 'bed sediment entrainment has neither been justified nor is it physically sound, because of the underlying empiricism and assumptions. The problem of determining bed sediment entrainment has actually not been solved but shifted to a consideration of the equilibrium near-bed concentration.

Note the generalized formulation by Cheng, which imposes the net flux of sediment exchange as the product of a parameter α and the difference between the spatially local and equilibrium near-bed concentrations – that is :

$$(5) \qquad \left(\omega C + \varepsilon_s \frac{\partial C}{\partial z}\right)_{z=a} = D - E = \alpha\,(C_a - C_{ae})$$

This formulation reduces to the assumption that bed sediment entrainment flux can be estimated using equilibrium near-bed concentration if α is set equal to the sediment effective setting velocity, whereas as $\alpha \to \infty$ it is equivalent to the premise that the local near-bed concentration is equal to the equilibrium value. In the former case, Cheng's formulation yields a Neumann boundary condition as equations (3) and (4) for sediment continuity equation. In the latter case it gives a Dirichlet boundary condition – that is :

$$(6) \qquad C_a = C_{ae}$$

According to Parker and Armanini and Di Silvio, the latter case (equation (6)) is not generally valid. This is contrary to Van Rijn who argues that both the Neumann and Dirichlet conditions can be used. Stated succinctly, equation (3) or (4) is the generally valid form, while equation (6) could at best be acceptable for special river processes.

A couple of examples of improper implementation of bottom boundary conditions can be provided herein. First, De Cesare attempted to model turbidity-induced sedimentation in reservoirs using the commercial CFX software for the 3D flow field and self-defined sub-model for suspended sediment. Apart from their problematic continuity equation for suspended sediment, the bottom boundary condition bears substantial ambiguity because the Neumann condition (equation (4)) was introduced, which is correct, and unnecessarily the bottom concentration was claimed to be 'extrapolated from upstream'. Second, in modelling sand deposition in the Colorado River in the Grand Canyon, Wiele used a depth-averaged 2D framework for the flow field, but a 3D sub-model for suspended sediment. The abnormal use of the logarithmic velocity profile over the flow depth does not produce the vertical velocity component that is however essential for the 3D sub-model of suspended sediment. Thus the inconsistency is evident. Apart from this, the implemented bottom boundary condition for suspended sediment is questionable. Wiele used equation (6) along with the Rouse formulation for suspended sediment concentration profile. At minimum, the use of the Rouse formulation is inappropriate as it is derived from the equilibrium condition that the fluxes of sediment deposition and entrainment are balanced (with negligible horizontal advection and turbulent diffusion). Obviously this is not the case in the pure deposition processes being studied.

3.2. Turbulent bursting-based bed sediment entrainment formulation

This lack of successful formulations of bed sediment entrainment continues to be one of the fundamental constraints precluding a clear understanding of sediment transport and also mathematical river modelling. According to the authors' experience, the modelling of a suspended sediment concentration field is sensitive to the bed sediment entrainment flux specified in the bottom boundary condition (equation (3)). Its influence is never trivial, even compared with that of the turbulent diffusivity ε_s for suspended sediment.

Physically, there are quasi-ordered coherent structures inherent in wall turbulent flows, which are characterised by the bursting processes consisting of deterministic sequences of intermittent and cyclic fluid motions referred to as ejections and sweeps. Perhaps in the last four decades or so since the work of Kline the major progress of turbulence research is the finding of the coherent structures and bursting characteristics. There has been a growing consensus about the turbulent bursting phenomenon as opposed to controversy. Reviews are provided by Robinson, Hussain and Cantwell. Bed sediment entrainment has been recognised to be controlled predominantly by turbulent bursting. A physically more appealing approach to bed sediment entrainment is accordingly to formulate it based on this mechanism. An encouraging formulation of this kind can be found in Cao and Hurther and Lemmin, although the development along this line is still in its early stage because of the use of time-averaged bursting variables. Further progress along this line is highly dependent upon the experimental techniques and instrumentation that can reliably measure the entrainment flux of bed sediment under a wide spectrum of hydraulic and sediment conditions, so that quality data are available to back formulations.

3.3. Reference elevation versus bottom boundary condition

The bottom boundary in 3D (and vertical 2D) mathematical models is set at a reference elevation $z = a$ above the real riverbed where turbulence is fully developed, and naturally the sediment flux is specified at this boundary. It must not be forgotten that even under the idealised situations of steady uniform flows, specifying the reference elevation can be an uncertain issue as stated by Zyserman and Fredsoe and Cao. Inevitably this issue always applies to models for generally occurring non-uniform flows. It is vital to recognise : (a) the lack of universally valid formulations of bottom boundary conditions for sediment ; and also (b) the lack of a comparative or sensitivity study of how the final computed sediment transport processes (concentration distribution, exchange flux across the near-bed boundary, and discharge over the channel cross-section, etc.) are influenced by the reference elevation.

3.4. Bottom boundary conditions for velocity and turbulent quantities

The above discussions have focused on the bottom boundary condition for sediment. Certainly the bottom boundary conditions for the mean velocity and turbulent quantities merit particular attention too. In turbulence modelling with the RANS and a closure model, the wall boundary conditions are almost exclusively based on the law of the wall for mean velocity, and on the additional assumption of local equilibrium of turbulence for turbulent kinetic energy and dissipation rate. No sediment effects have been incorporated in these formulations. Moreover, it is recognised that sediment concentration is normally higher nearer the bed, and its strong interaction with local turbulence is never negligible. However, this has rarely been studied, rendering further uncertainty with mathematical river modelling.

4. MISUSE OF MODEL CALIBRATION AND VERIFICATION/VALIDATION

It is well-known that the governing equations of deterministic mathematical models for alluvial rivers are open systems, and various empirical constitutive relationships have to be incorporated to close the models. The practice of applying these models involves two separate stages, namely calibration and verification/validation. In the first stage (calibration) the various empirical parameters are adjusted so that the model reproduces results in agreement with measurements on the river being studied. In the second stage (verification/validation) the model, along with the empirical parameters calibrated in the first stage, is run on a separate dataset for the same or similar river problem. If the calibrated model, without resorting to further adjustment of parameters, reproduces the measured data with acceptable error tolerance, then it is widely considered as verified or validated. Rarely can one find examples in the literature of models not verified or validated.

4.1. Model calibration

Whilst this two-stage calibration-verification process has been widely employed in many other areas, some basic problems with respect to fluvial applications have been ignored. A mathematical river model encompasses a number of parameters to be determined. One primary question is whether there is a unique combination of these parameters. From time to time the same (or similar) results are produced using different sets or combinations of model parameters. Usually there is no way to choose between these sets of parameters, other than to invoke extra-evidential considerations such as symmetry, simplicity, flexibility, personal, political or metaphysical preferences as well as prejudices and financial considerations. A secondary question arises as to how the overall performance of modelling can be objectively judged in comparison with measurement. This is especially critical for 3D modelling as normally there are many megabytes of numbers (typically with over 50000 nodes for a real river problem), and it is almost impossible for model developers and end-users to view, assimilate, interpret and present even a small fraction of the output. That way, the judgment of acceptable agreement with measured data is virtually on a basis of a limited portion of information, for example some selected verticals and cross-sections, etc. Known to many model developers and end-users is the fact that it is fairly feasible to reconcile the computed results to measurements within a local area by tuning the various parameters. Thirdly, it is hard to specify the initial conditions, whereas the computation can be sensitively influenced by the initial status in the non-linear systems ; therefore the agreement between computed and measured results in general is largely not unbiased but subjective.

4.2. Model verification and validation

As for as the second stage (verification and/or validation) is concerned, it is worthwhile to note the arguments by Oreskes in their review of earth science modelling. They claim that verification and/or validation of numerical models of natural systems is impossible because of logic considerations on a philosophical basis. Konikow and Bredehoeft argued similarly for numerical models of groundwater problems. According to Oreskes both verification and validation are affirmative terms. Strictly, verification is an assertion or establishment of truth. A verified model is thus useful as a prediction tool because of its demonstrated truth, and implies its reliability as a basis for decision-making. Equally correct is the term 'validation', which usually connotes legitimacy. It can, but does not necessarily denote an establishment of truth. Instead, it indicates the establishment of legitimacy, generally in terms of contracts, arguments, and methods. Validation means making legally valid, granting official sanction to or confirming the validity of something. A valid model contains no known errors or detectable flaws and is internally consistent.

Verification is only possible in closed, rather than open systems, in which all components of the system are established independently, and are correct. Its application to natural systems is misleading. Alluvial river models are never closed systems, and therefore it is incorrect to use the term 'verification' for such models. Below are two specific reasons that make alluvial river models open.

First, the model requires a number of input parameters that are not completely known. These input parameters are often embedded in turbulent closure modules, boundary conditions, sediment transport and entrainment functions as well as numerical discretisation schemes, etc. Second, the observation and measurement of both independent and dependent quantities are laden with inferences and assumptions. Although many inferences and assumptions can, in some cases, be justified with experience, the degree to which the assumptions hold in new and complicated studies can never be established a priori. Alluvial river systems are complicated in that turbulence is one of the last problems in classic physics, which remains to be solved, and this is further aggravated due to the presence of sediments.

It is essential to recognise the restricted sense of the term 'validation'. Legitimacy, official sanction, or being free of apparent errors and inconsistency does not necessarily mean truth or correctness, although truth or correctness is not precluded. It is misleading if the term validation is used to refer to actual modelling results in any particular realisation. It is fairly popular for river modellers to use interchangeably the terms verification and validation. Thus they misleadingly imply that validation establishes model veracity. Even more critically, the term validation is used to suggest that the physical river phenomenon is accurately represented by numerical models.

As stated above, there exist a lot of critical problems with the model calibration-verification/validation phases, both logically and practically. The most significant problem comes with the verification/validation phase, where the model is claimed a success. This is, as a matter of fact, committing the basic logic error of affirming the model output. Oreskes describes this as follows :

> 'To claim that a proposition (or model) is verified because empirical data match a predicted outcome is to commit the fallacy of affirming the consequent. If a model fails to reproduce observed data, then we know that the model is faulty in some way, but the reverse is never the case.
>
> Confirming observations do not demonstrate the veracity of a model or a hypothesis, they only support its probability'.

The misuse of the terms verification and validation in mathematical river modelling can be risky with respect to public interests. Often the decision-makers may not be experts in river hydraulics. It is the responsibility of model developers and end-users to correctly inform the decision-makers of what mathematical models can realistically reflect, and more essentially the degree to which the modelling results can be relied upon.

4.3. Are mathematical river models useful ?

The above comments do not constitute the authors' intention to reject the use of mathematical river models. The criticisms advanced in this overview are aimed to help stimulate a wider awareness of the limitations of such models. Mathematical river modelling is nothing but a semi-empirical approach to river problems, which embeds enhanced understanding of the physics of the problem and is therefore more advanced than earlier crude, yet simpler, methods. Human society has benefited greatly from empirical approaches. The more experience we get, the better mathematical river modelling can become. The greater the number and diversity of confirming observations by mathematical models, the more probable that the mathematical models are not flawed. However. Mathematical river modelling must be appropriately used and recognised ; overestimation of its capability is misleading the public and can carry risks for society. The common usage of the so-called calibration-verification/validation process is often nothing more than a self-evident statement that an acceptable match between model results and observations is obtained by tuning the various parameters. Rarely do mathematical river models have the predictive role that is frequently claimed. There is no guarantee that the calibrated models will reproduce results in agreements with measurements for other independent river problems.

5. STRATEGY FOR IMPROVED PRACTICE OF MATHEMATICAL RIVER MODELLING

In the long-term sense, enhanced understanding of the physics of fluvial sediment interaction with turbulence is the key to improved mathematical river modelling. Given its fundamental nature, any breakthrough in turbulence appears to be far into the future. However, with the current knowledge of fluvial hydraulics and sediment transport, there is wide scope for strategic studies to improve the practice of mathematical river modelling. Drawn from the various (but not all) components of such models addressed above, the following aims can be realised in the immediate future so that best practice guidelines can be established for use by model developers and end-users such that optimum decision-making can be delivered.

5.1. Assess various model components

As addressed earlier, various model components contribute to the uncertainty of mathematical river modelling. These comprise sediment transport functions, resistance relations, turbulence closure models, bottom boundary conditions, asynchronous solution procedures, and numerical algorithms etc. Some of these have been apparent since the beginning of the modelling practice, yet their quantitative impacts remain to be identified and assessed. As already argued above, these components must be evaluated in isolation.

There have been a number of sediment transport functions derived on the basis of various theories and observed data. From existing marketed river software, often the potential users have a pool of sediment transport functions to choose from for their specific project purposes. Distinct sediment functions normally yield quite different sediment discharges and bed evolution. During the post-assessment phase modelling, it is the responsibility of modellers to judge what to rely on, based on their experience and preference. This status calls for considerable expertise and physical insight into fluvial sediment transport and also into the design of the software in question. At the same time, this status characterises the need for a comprehensive assessment of a range of sediment functions applied to different situations. Through such an assessment of sediment transport functions, a general guideline is likely to be established for use by practising modellers and engineers.

The formulation of the bottom boundary conditions is also one of the major sources of model uncertainty. For instance, there are various relationships for the equilibrium near-bed concentration defined at different reference heights above the bed. Yet their respective performance, when applied in vertical 2D and 3D models, has never been assessed. Likewise, the bottom boundary conditions for mean velocity, turbulent kinetic energy and dissipation rate, based on the law of the wall and the additional assumption of local equilibrium of turbulence, have never been examined when applied to sediment-laden flows.

The asynchronous solution procedure, commonly utilised in river modelling, has rarely been studied, the inherent effect of which remains unclear. In a recent study by Cao it is shown to result in inaccuracy comparable to that of a largely tuned friction factor, or even lead the physical process mathematically ill-posed. However, this study has been limited to 1D laboratory scales. Extension to practical river processes is of importance.

It is necessary to define a set of benchmark river processes with reliable and systematic observed data, against which the various model components can be assessed. Clearly, this requires expert knowledge of river processes, including a comprehensive background in the basic disciplines of fluid mechanics, sediment transport mechanics, numerical computing, and software design. There seems to be no doubt that defining the benchmark river processes is technically feasible.

5.2. Evaluate model performance by quantitive performance index

Subject to the primary purposes of specific modelling applications, the evaluation of various model components should be performed with a sensibly defined 'performance index' because the outputs of modelling can be more or less sensitive to the model components. For instance, sediment discharge is often more sensitive to the sediment transport function implemented in the modelling than the bed evolution. Accordingly, if sediment discharge or concentration field is of importance, then it is of priority in defining the performance index. This case pertains to modelling applications biased towards the ecological aspects of the fluvial environment as linked to fine sediment concentration. In river engineering applications, the morphological development is usually of interest. Then it should be considered as the primary factor in evaluating model components. The normalised L_1 norm between changes of bed elevation, defined in equation (2) of the companion paper, is just an example of a performance index. In 2D and 3D spaces, it can be defined similarly. The 'performance index' could be more or less similar to the 'applicability index' proposed by Williams and Julien for assessing sediment functions. However, the former is a generalised index that can be applied to measuring modelling quality in response to various model components (not only sediment functions, but resistance relations, turbulence closures, bottom boundary conditions, asynchronous solution procedures etc.).

Defining an overall quantitative performance index (or indices) is sensible and often imperative in mathematical river modelling, especially in 3D cases. Echoing earliar statements, model calibration and confirmation can be subjective and are insufficient if only a fraction of the modelling outputs are examined. Nevertheless, in current modelling practice, this kind of overall performance index has rarely been defined and properly delivered, even though it is frequently claimed that modelling matches observations or other modelling or analytical results. The use of a performance index (or indices) can help normalise model calibration and confirmation.

5.3. Implement consorted action and professional training

To improve the practice of mathematical river modelling, consorted action of the academic and practising communities is vital. The complex nature of river processes determines that more active exchanges and sharing of specialist knowledge and data between academics and practising engineers (as model developers and end-users) are essential. These can, in due course, promote the application of the latest fundamental research achievements in modelling practice, and conversely stimulate the awareness by academics of the real world of river processes. The lack of in-depth understanding, by model end-users and those who rely purely on model outputs to make decisions, of the basics of river processes implies a distinct training need for professionals if improved river modelling is to be delivered.

6. CONCLUSIONS

Full 3D modelling is the way forward for alluvial river processes. The major impediment to this advanced modelling practice is the lack of well-grounded turbulence closure models and bottom boundary conditions (for suspended sediment and mean flow quantities as well). These issues are rooted in the core of the discipline of fluvial sediment transport, but have been underappreciated in the current modelling practice.

The calibration and verification/validation methodology commonly used in mathematical river modelling is problematic. Model performance is overstated by using the affirmative terms 'verification' and 'validation' which can mislead the public and decision-making. What is really revealed by both calibration and verification/validation is essentially showing, or at best confirming, specific cases where models, along with a combination of selected parameters, produce result matching observations. To claim a predictive role of mathematical river models can be risky both currently and in the foreseeable future. This is particularly the case when the predictive role is judged based on fractional agreement between modelled results and measurements without the use of overall performance indicators.

Strategic studies are technically feasible to improve the mathematical river modelling practice within the current knowledge framework of river mechanics. A set of benchmark river processes need to be defined, against which various model components such as sediment transport functions, resistance relations, simplified continuity equations, asynchronous solution procedures, bottom boundary conditions, and turbulence closure models can be assessed individually. It follows that a systematic set of guidelines can be established to norm the mathematical river modelling practice, and subsequently optimize modelling-based decision-making.

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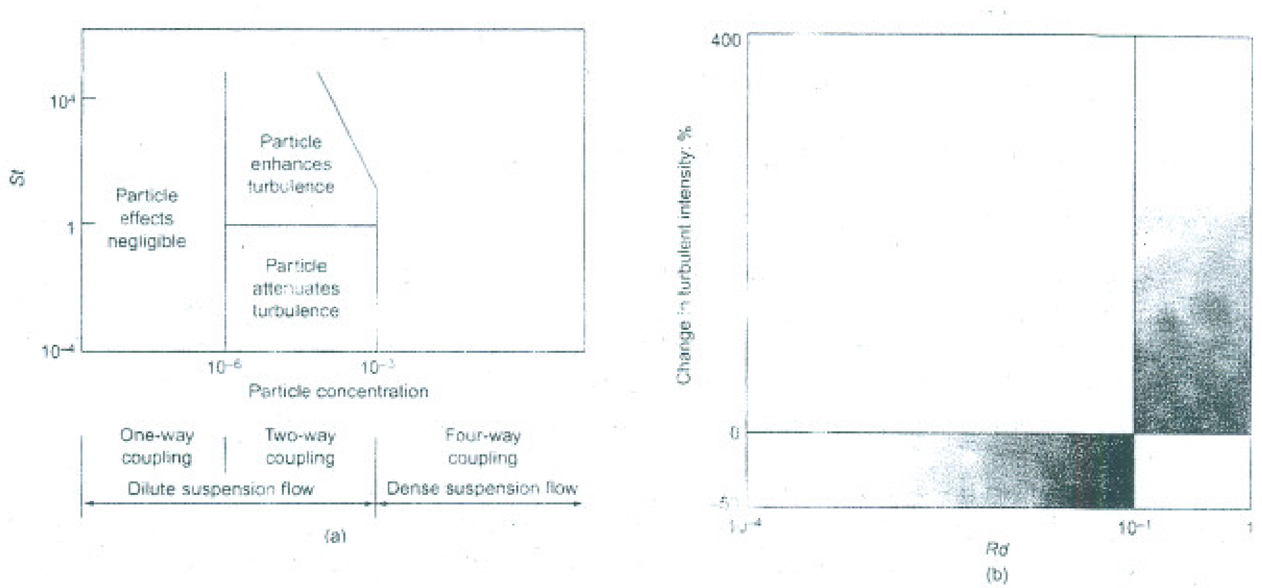

Fig. 1. Illustration of particle effects on turbulence : (a) particle-turbulence interaction as influenced by the particle Stokes number and concentration : and (b) effect of relative particle size on turbulence intensity. (The greater grey level in the filled area illustrates larger density of data symbols distribution).

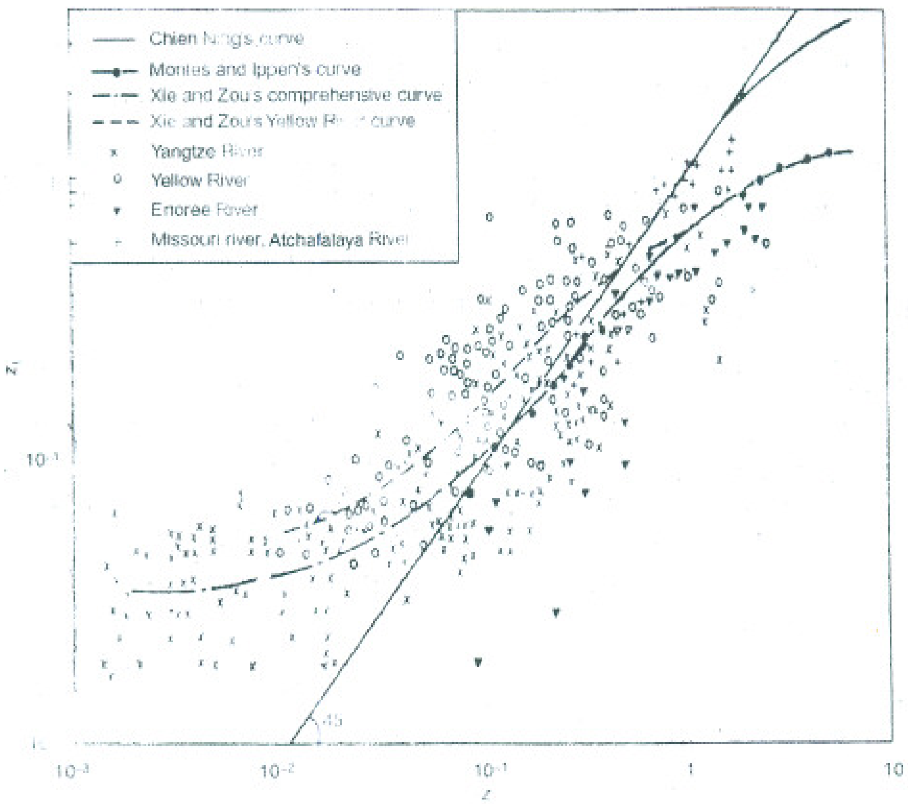

Fig. 2. Relationship between z and z_1 (adapted from Xie and Zou)

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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	15
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