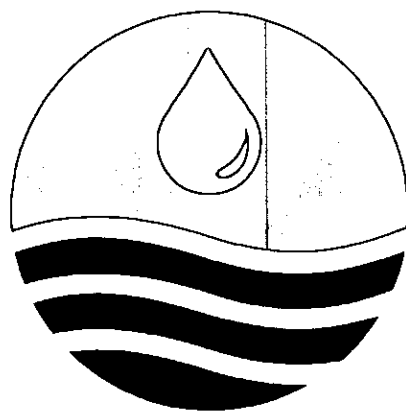




水庫沉滓運移模式研究與應用(1/2) —以石門水庫爲例

Investigation and Application of the Sediment
Transport Model in Reservoir (1/2)
—for Shih-Men Reservoir



主辦機關：經濟部水利署

執行單位：國立交通大學防災工程研究中心

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並編修 NETSTARS 模式英文使用手冊。

本年度研究計畫中完成主要工作分別有；

1. GSTARS 3.0 模式評估測試。
2. 水庫沉滓運移數值模式發展與測試。
3. NETSTARS 模式英文版使用說明編撰。
4. 協辦中美水庫淤積技術支援合作計畫。

第一章 緒論

1.1 計畫緣起

台灣地區山高坡陡，河川流短湍急；降雨量雖然豐富，但豐枯懸殊，因此水資源之利用常以建造水庫調節水量。但台灣地區因地質條件不佳，山區地質鬆軟，再加上多年來山坡地的大量開發，水土保持工作未落實，每逢颱風暴雨侵襲，山洪自上游挾帶大量土石奔馳而下，淤積於庫內，造成水庫在蓄水的同時也兼蓄砂。根據水資會 83 年的統計資料，台灣水庫每年的泥砂淤積總量高達 14,600,000 m³，相當於一座明德水庫之設計有效容量。在優良壩址有限，復以工商發展、土地價值不菲，及環保、生態意識高漲，使得構建大型水庫以獲得新水源的開發方式，必須付出高昂的土地成本與社會成本，而日益降低其作為開發新水源方式的可行性。因此，如何減少水庫淤積，以延長現有水庫壽命，並充分利用有限的水資源，實為當務之急。為了保持庫容，減少或清除水庫淤積，首先必須研究入庫泥砂在水庫內之運移、沖淤和分佈的規律，而數值模擬即是最佳的工具之一。數值模擬具有預測的功能，可輔助解答水庫淤砂在規劃、設計及運行階段所面臨到的問題。因此，考量台灣水庫淤砂的特性，發展適用於台灣地區水庫沉滓運移數值模式，配合實際案例之模擬，以具體分析台灣水庫淤砂的相關課題，不失為值得探討的方向。

1.2 模式發展需要性分析

考量電腦計算時間與資料率定驗證的問題，目前在水利工程相關課題的實用分析上，一般多採用二維、擬似二維及一維的數值模式。基本

5. 協辦中美水庫淤積技術支援合作計畫。

本計畫執行迄今，已如預期完成上述所有的工作內容，執行成效符合預期。

1.4 報告內容

本計畫報告內容共可分為七章，第一章為緒論，說明本計畫之緣起、目的、執行成效及報告內容；第二章為石門水庫基本資料蒐集與分析，敘述分析石門水庫的相關資料與其所面臨的問題；第三章為模式評估分析，評估的模式包含水庫沉滓運移模式、GSTARS 3.0 模式及 NETSTARS 模式；第四章為 GSTARS 3.0 模式評估測試，根據該模式之功能，利用美國內政部墾務局所提供的案例，進行模式的測試工作；第五章為水庫沉滓運移模式之發展與測試，重點在於描述模式之發展與案例測試工作；第六章為水庫沉滓運移模式與 GSTARS 3.0 模式比較分析，針對石門水庫的特性，選定幾個適當案例進行模擬比較分析；第七章為 NETSTARS 模式英文使用手冊；第八章為結論與未來工作。

第二章 石門水庫基本資料蒐集與分析

2.1 水庫概述

石門水庫位於桃園縣龍潭鄉之大漢溪主流上，集水面積 763.4 km²。水庫於民國 45 年 7 月開工，民國 52 年 5 月開始蓄水，至民國 53 年 6 月施工完成。石門水庫為一多標水庫，具有防洪、灌溉、給水及發電功能。大壩位於大漢溪之石門峽谷，為一滾壓式土石壩，壩頂標高為 252.1 m，最大壩高 133.1 m，壩頂長 360 m，壩身體積 7,059,000 m³。水庫滿水位標高為 245 m，呆水位標高 195 m。滿水時庫區水域長達 16.5 km，周長 41 km，面積 8.15 km²。民國 53 年完工時之原始總容積為 309,120,000 m³，呆容積為 5,724,000 m³，有效容積為 251,880,000 m³。

2.2 集水區地勢、水系

石門水庫集水區位於雪山山脈之西北部，略呈南北向之長腰形，地形大致自西北向東南逐漸增高，西北部為丘陵地帶，東南部為山岳地帶。本集水區標高自最低之石門水庫壩址 135 m 至最高之品田山 3,529 m，平均高度為 1,752 m。雪山山脈主峰一雪山(3,884 m)，在大漢溪源頭之西約 5 km。自雪山向東北綿延至品田山，始分歧兩支山嶺線，圍成石門水庫集水區，此兩支山嶺線為與東面蘭陽溪。東北新店溪、西面頭前溪之分水嶺。

大漢溪為淡水河第一大支流，發源於品田山北麓之泰崗溪，自發源地曲折北流，流至秀巒之北，與源出大壩尖山北麓之白石溪匯合北流後稱為玉峰溪，至高台右折東北行，溪谷稍寬，水流湍急，復於三光之東北納源出崩崩山之三光溪，再左折北行乃稱大漢溪，數度橫切山嶺，先後納卡拉溪於蘇樂，黑白庫溪於高義，匹亞溪於榮華，色霧閣溪於色霧閣，高坡溪於高坡，義盛溪於羅浮，霞雲溪於霞雲坪，流量漸增，至角板山又左折西

石門水庫集水區之出露地層，可以屈尺斷層為界，分為東南側雪山山脈帶之古第三紀岩層及西北側屬西部麓山帶之新第三紀岩層。水庫集水區地質分佈如圖 2-2 所示。古第三紀岩層分佈於集水區之東南側，屬輕度變質岩層，岩性以硬頁岩、板岩及砂岩為主。新第三紀岩層分佈於集水區之西北部，包括庫區周邊地帶，岩性以砂岩、頁岩為主。第四紀地層分佈於大漢溪主支流沿岸地區及地勢較平緩之台地山麓地帶，膠結程度低，易受風化沖刷。整體而言，集水區岩性脆弱，劈理、節理及裂縫發達，且經強力擠壓而破碎，加以地勢陡峻，暴雨時易遭沖蝕。

石門水庫集水區之土地利用以國有林班地 530.24 km^2 為最廣，佔全集水區面積之 69.46 %; 山地保留地 167.88 km^2 居次，佔 21.99 %; 私有地 23.5 km^2 居第三，佔 3.10 %; 其他用地 41.58 km^2 ，佔 5.45 %。其中以山地保留地為集水區人為活動最頻繁地區，因人口急速增加，土地需求殷切，更因溫帶果樹之引進，致超限利用嚴重，泥砂產量之大增，影響水庫淤積及水質最大。

2.5 水文站

石門水庫集水區現設有 16 個雨量站，自上游依序為西丘斯、鎮西堡、白石、秀巒、鞍部、玉峰、三光、嘎拉貿、巴陵、高義、霞雲、復興、長興、石門、十一份、池端等。圖 2-3 為石門水庫集水區雨量站位置圖，表 2-4 為石門水庫集水區雨量站概況表。

石門水庫集水區現設有秀巒、玉峰、稜角、高義、霞雲等 5 個流量站，低水時利用流速儀求算流量，高水時則利用浮標法求算流量。石門水庫壩址站則利用水庫運用演算，以推算流量資料，圖 2-4 為石門水庫集水區流量站位置圖，表 2-5 為石門水庫集水區流量站概況表。霞雲站位於所有水文站的最下游，控制水庫集水區面積 80 % 以上，因此最為重要，設有自計水位

陳樹群等(2001)亦以集水區面積為指標，針對全省 20 座中小型水庫集水區進行產生模式推估，並以全省 33 座水庫集水區為對象以作比較，經回歸分析後可得

$$\text{中小型水庫 } SY = 0.235A^{1.193} \quad (2-4)$$

$$\text{全省水庫 } SY = 0.4241A^{0.8982} \quad (2-5)$$

式中， SY 為年平均淤積量($10^4\text{m}^3/\text{yr}$)。

林俊輝(1984)由石門、德基、曾文與霧社水庫所建立之產砂公式為

$$q_s = 0.013416P^{3.26}C^{-1.43}S^{3.23} \quad (2-6)$$

式中， q_s 為年泥砂產量($\text{ton}/\text{km}^2\text{-yr}$)； P 為年平均雨量(以 1000 mm 為單位)； C 為森林覆蓋率(%)； S 為平均坡度。

林長立(1985)利用石門水庫之實測資料推估集水區泥砂產量公式為

$$\log q_s = -7.57 + 1.54 \log P + 5.54 \log S - 29 \log C \quad (2-7)$$

蔡長泰(1992)將 1963 年前之石門水庫各子集水區年泥砂產量與洪峰流量進行分析探討，求得

$$SE = 0.0069Q^{0.661} \quad (2-8)$$

式中， SE 為年淤砂量(mm)； Q 為年洪峰流量(cms)。

中華水土保持學會(1989)推估石門水庫之集水區年平均產砂模式為

$$q_s = 10^{-7.75} P^{1.54} S^{5.54} C^{-29} \quad (2-9)$$

式中， q_s 為年泥砂產量($\text{ton}/\text{km}^2\text{-yr}$)； P 為年平均雨量(以 1000 mm 為單位)； C 為森林覆蓋率(%)。

中華水土保持學會(1989)將石門水庫集水區劃分為 6 個子集水區，並考慮會影響集水區產量的因子(如地貌係數、覆蓋率等)，集水區之產砂推估模式可表示為

之粒徑分佈則較為忽略，一般文獻中僅能蒐集少數水庫之資料，且資料亦不完整。就石門水庫而言，粗顆粒沉澱會沿程落淤於庫區中上游段，粒徑介於 0.05~3 mm，一般歸類於非凝聚性沉澱；壩址前的淤砂多為細顆粒沉澱，粒徑介於 0.003~0.05 mm，一般歸類於凝聚性沉澱(cohesive sediment)的範疇內。

由上述水庫泥砂的淤積行為與淤砂沉澱特性可知，石門水庫泥砂的重要特性為

1. 非均勻質：包含中上游段的粗顆粒沉澱與下游段的細顆粒沉澱。
2. 凝聚性沉澱：主要為壩址前的淤砂部分。
3. 非平衡輸砂：細顆粒沉澱於庫區內的沉積行為會有時間與空間上的延滯效應，即所謂的非平衡輸砂行為。

2.8 水庫淤積狀況

石門水庫初始規畫設定時，原估算平均年泥砂淤積量為 800,000 m³，但運轉後之淤積測量結果顯示實際淤積率遠超過此設計值。石門水庫幾乎每年均實施一次淤積測量，表 2-6 為石門水庫歷年淤積記錄。依表 2-6 所列之累積淤積量，至民國 66 年 11 月之年平均淤積量為 2,893,000 m³，為規畫設計值之 3.6 倍；至民國 88 年 11 月止，累積淤積量已達 58,810,000 m³，致水庫總容量損失約 19 %，年平均淤積量為 1,610,000 m³，相當於原規畫設計的 2 倍。另外，表 2-6 中部份年度之淤積量呈現負值，這可能是測量誤差與未記入清淤量所致。歷次測量所得之淤積量以民國 52 年 5 月至 53 年 3 月之 19,470,000 m³ 居首，此乃由於民國 52 年 9 月 9 日至 14 日葛樂禮颱風過境帶來豪雨(64 小時最大降雨量達 1,375 mm)，集水區崩塌嚴重，土壤大量流失之故。民國 84 年 11 月至 85 年 11 月間 8,670,000 m³ 之高淤積量乃因 85 年 7 月 30 日至 8 月 2 日賀伯颱風過境之故，集水區 24 小時最大降雨量

km 範圍庫區之淤泥。石門水庫之水力抽泥設施包括工作船、沉水式抽泥泵、高壓噴水泵、主輸泥泵、輸泥管線及沉澱池。圖 2-5 為工作船及附屬設施之示意圖。

進行抽泥作業時，先以高壓噴水泵攪鬆庫底淤泥，再以沉水式抽泥泵抽取泥水，經主輸泥泵及輸泥管線，送至沉澱池。石門水庫之沉澱池設於後池堰下游右岸之河川地，共有 13 座，總容量約 4,200,000 m³，個別長度介於 200 至 300 m，寬度介於 100 至 200 m，深度為 5 至 10 m，其佈置如圖 2-6 所示。輸泥管線之泥水由 12 英吋閘門導入沉澱池上游側之進水渠，再經溢流堰均勻流入沉澱池。尾水則經沉澱池下游側之溢流堰洩入出水渠，再經排水系統注入大漢溪。

石門水庫水力抽泥船清淤於民國 74 年 3 月由得標廠商正式作業，預定每年抽泥 300,000 至 450,000 m³，淤泥抽至沉澱池沉澱 20 天後再計量付費。合約同時規定抽泥船所抽出泥水之含砂濃度不得低於 20%，以減少清淤水量之損失。至民國 84 年 4 月，水力抽泥船共清淤 3,722,000 m³，發電進水口前之淤積面標高由原來之 174.55 m 降至 162.35 m，已低於擋泥牆之牆頂 12 m，壩前 1.5 km 抽泥範圍內之淤積面亦平均下降 7.2 m。

石門水庫水力抽泥之成本包括抽泥與沉澱池兩部分，根據民國 81 年之資料，該年共清淤 300,000 m³，抽泥船作業成本為 60,265,000 元，沉澱池作業成本為 20,964,000 元。依此，抽泥與沉澱池之單位成本分別為每立方公尺 201 元及 70 元，合計 271 元。

2.10 本計畫擬解決之問題

藉由上述石門水庫基本資料與現況問題之整理分析，本計畫在石門水庫應用例部分，擬定計畫執行方向為庫區浚渫方案之評估。茲將此浚渫方案評估條列如下

提供與數值模式作為上游入砂量之設定依據。

5. 壩址水位：石門水庫庫區的水位資料，可作為模式下游水位邊界條件設定之依據。

綜觀上述之分析，目前所蒐集的水庫相關資料，應可滿足數值模式所需的輸入資料。並可進一步進行相關案例的模擬探討。

2.11.2 資料率定與驗證

為合宜的利用數值模式探討石門水庫淤砂的相關課題，首先需先利用歷史資料對數值模式進行率定與驗證的工作，以推求適用於石門水庫的相關模擬參數。在輸砂模擬部份，用以率定的依據為床形變化資料。石門水庫每年均有施行淤積測量，即每年均有床形變化的資料，但水庫自民國 62 年起開始於中游段庫區以挖泥船清淤，其後又於民國 74 年起在下游段庫區實施水力抽泥船清淤以控制發電進水口附近之淤積面，因此，有必要針對此清淤量對模式率定驗證的影響進行評估分析。

1. 就石門水庫的清淤記錄而言，雖然最早於民國 62 年即開始在中游段庫區實施清淤，但此時實際需求量不大，所以直至民國 66 年才算真正開始有較大量的清淤工作。因此，民國 66 年前底床淤積資料應可視為不受水庫清淤之影響。
2. 石門水庫於民國 74 年起在下游段壩址前庫區實施水力抽泥船清淤，以控制發電進水口附近之淤積面。水庫壩址處可視為入庫泥砂運移的終點，且次處的淤泥屬於細顆粒沉澱的範疇，淤積行為屬於靜水沉積，因此在此處施行浚漂對底床高程的改變應該只會造成局部的影響，對於模式率定驗證影響應也僅止於近壩址處的若干區域。
3. 根據表 2-6 水庫歷年淤砂資料，其中 73.11~74.11 及 84.11~85.11 兩次測量的淤砂量特別大，分別為 $3,695,000 \text{ m}^3$ 及 $8,670,000 \text{ m}^3$ 。根據表 2-7 之清淤量記錄，74 年的清淤量為 $556,844 \text{ m}^3$ ，清淤量約為當年水庫淤積量

表 2-1 石門水庫集水區地文特性表

項目	符號	數量	單位	備註
集水區面積	A	763.4	km ²	
集水區周長	P	156.96	km	
河川總數	N	64		
本流長度	Lo	94.01	km	
本支流總長	L	352.63	km	
集水區平均寬	W	8.12	km	$W=A/Lo$
河流長與集水區平均寬之比	R	11.577		$R=Lo/W=Lo^2/A$
集水區形狀係數	F	0.086		$F=A/Lo^2$
集水區密集度	C	0.623		$C=2\sqrt{\pi A}/P$
河川密度	Ds	0.084		$Ds=N/A$
排水密度	Dd	0.462		$Dd=L/A$
集水區平均高度	E	1,417.12	m	
山嶺線平均高度	H	1,752.00	m	

表 2-2 石門水庫之月蒸發量統計

單位：mm

月份	一	二	三	四	五	六	七	八	九	十	十一	十二
蒸發量	56.33	51.16	62.46	73.72	90.03	103.4	137.8	127.2	112.6	112.2	85.47	69.23
總計	1081.55											

表 2-5 石門水庫集水區流量站概況表

站名	東經	北緯	標高 (m)	流域面積 (km ²)	設站日期 (年/月)	備註
秀巒	121° 17'	24° 37'	827	115.93	45/8	
玉峰	121° 18'	24° 40'	701	335.29	45/8	遙報站
稜角	121° 22'	24° 41'	525	107.76	45/7	
高義	121° 21'	24° 43'	438	542.03	46/1	
霞雲	121° 21'	24° 48'	249	622.8	51/1	遙報站
石門	121° 14'	24° 49'	137	763.4	35/3	遙報站

表 2-7 石門水庫中游段庫區挖泥船歷年清淤量

年度	清淤量 (m ³)	累計清淤量 (m ³)
66	152,795	152,795
67	123,203	275,998
68	194,929	470,927
69	165,063	635,990
70	224,376	860,366
71	197,732	1,058,098
72	70,649	1,128,747
73	219,901	1,348,648
74	556,844	1,905,492
75	426,475	2,331,967
76	300,315	2,632,282
77	385,452	3,017,734
78	264,730	3,282,464
79	281,700	3,564,164
80	835,000	4,399,164
81	385,645	4,784,809
82	915,845	5,700,654
83	308,404	6,009,058

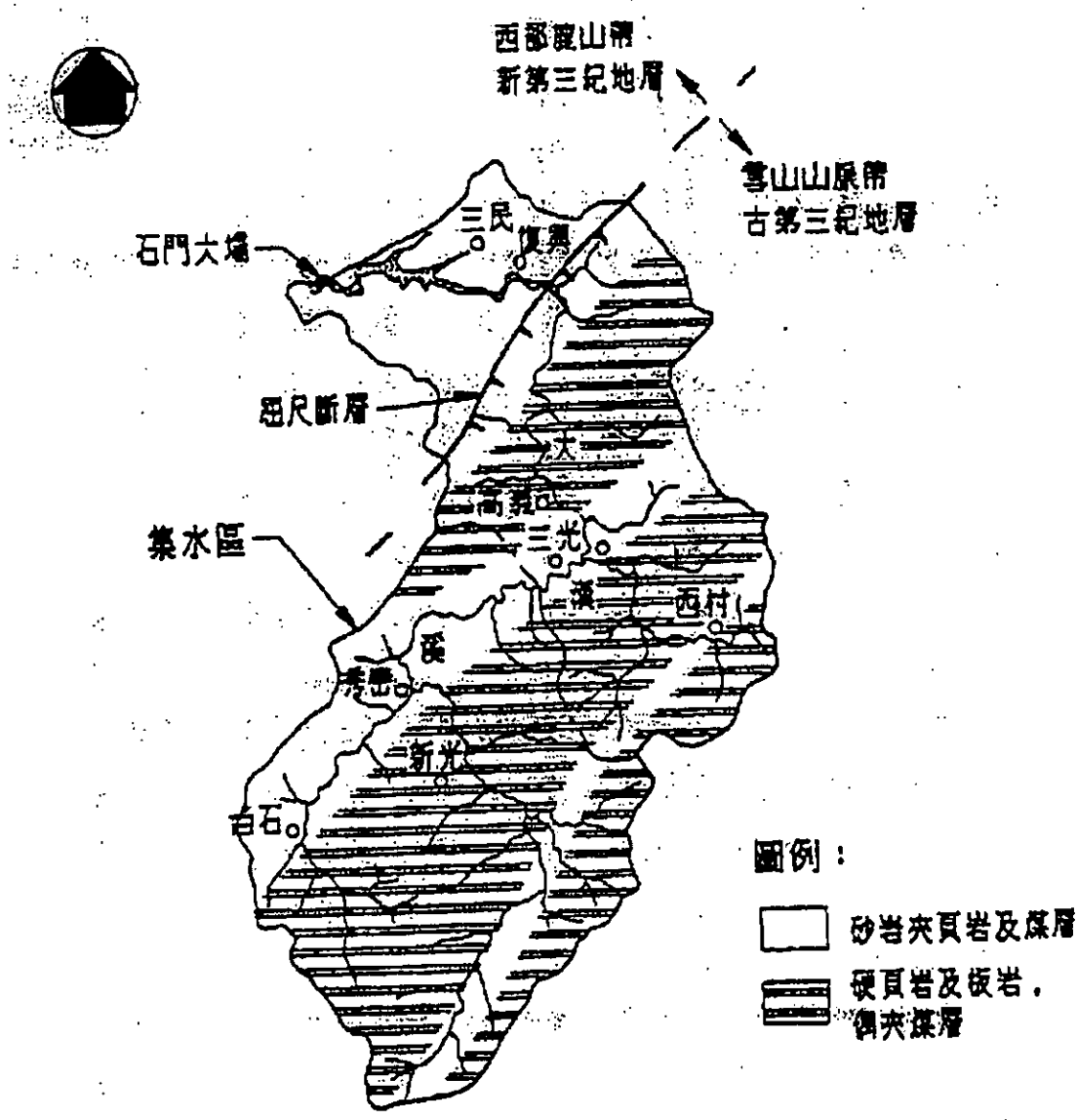


圖 2-2 石門水庫集水區地質分佈圖

資料來源：「石門水庫泥沙控制」，中興工程第五十八期

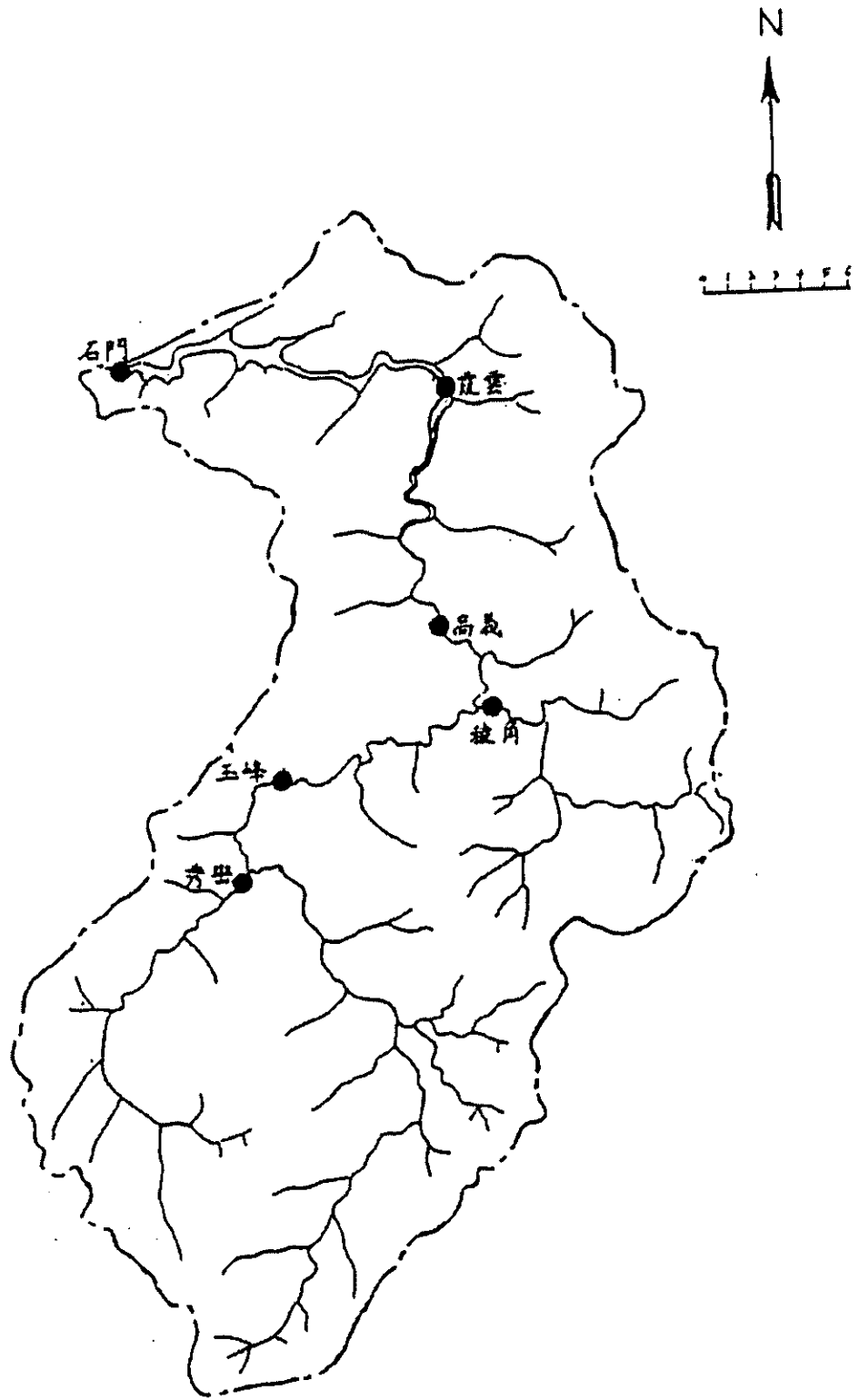


圖 2-4 石門水庫集水區流量站位置圖

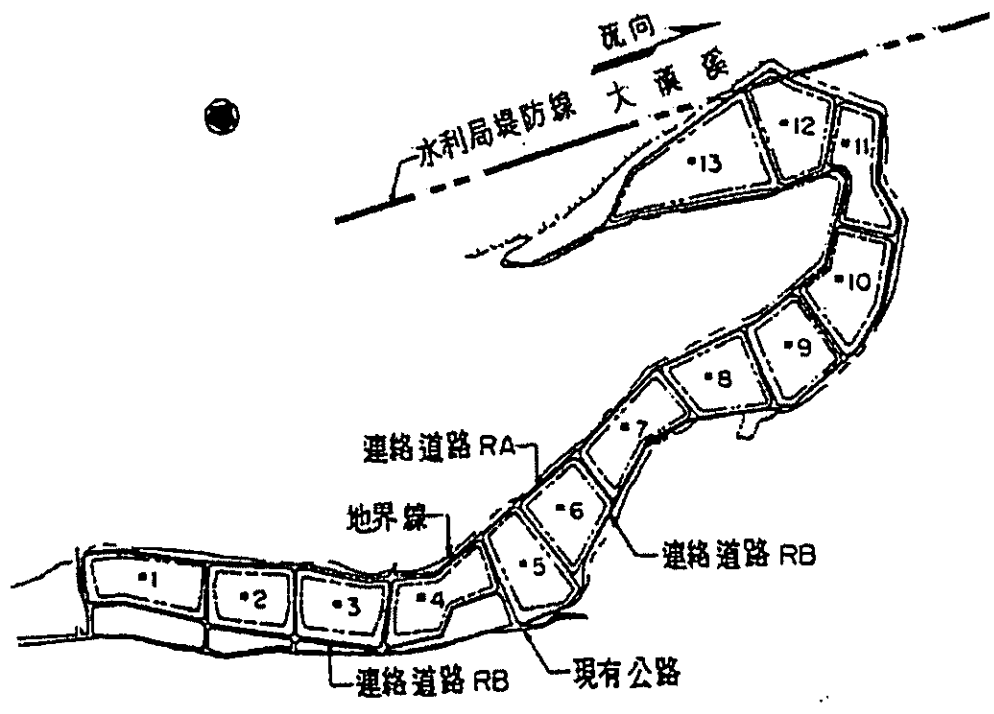


圖 2-6 石門水庫沉澱池平面佈置圖

資料來源：「建立水庫高效率排砂」，經濟部水資源局(2000)

第三章 模式評估分析

本計畫在石門水庫應用例部分，擬定計畫執行方向為庫區浚渫方案之評估。為達此標的，模式的研選方向為模式需能模擬水流與輸砂機制在平面的變化情況。三維模式在資料檢定與驗證上仍有不足，且計算仍嫌龐大，在實用上仍有其困難存在。因此，為模擬石門水庫的浚渫相關課題，模式研選方向應為擬似二維模式與水平二維模式。

GSTARS 模式為擬似二維輸砂模式，已成功應用於美國地區許多實際河川問題之模擬，因此，本計畫在擬似二維模式部份，選用 GSTARS 模式，作為模式比較分析的基礎。本計畫的主要工作項目之一為發展水平二維模式，此模式(水庫沉滓運移模式)即可作為模式比較分析中二維模式的基礎。另外，編撰 NETSARS 模式英文使用手冊亦列為本計畫之工作項目，因此，在模式評估時，亦將 NETSTARS 模式納入考量。

3.1 水庫沉滓運移模式

(一)模式發展沿革

本模式為國立交通大學土木系楊錦釗教授累積多年的研究成果，期間承蒙多個單位的經費補助，使本模式之發展能漸趨完整。迄今，該模式曾分別應用於淡水河、景美溪截彎取直工程及阿公店水庫更新改善工程之排砂模擬工作。目前模式發展的重點在於水庫淤砂模擬，以探討石門水庫相關的淤砂課題。

(二)模式功能

茲將本模式重要功能與特性敘述如下：

1. 模式採用正交曲線座標系統(貼壁座標系統)之控制方程式，對於天然渠道堤線不規則變化之幾何形狀可沿著河道邊界建立計算格點，一方

3.2 GSTARS 模式

(一)模式發展沿革

GSTARS 模式 (Generalized Stream Tube model for Alluvial River Simulation)發展迄今十餘年，已成功應用於美國地區許多實際河川問題之模擬。該模式不但具備一般一維沖淤河川模擬之功能，利用流管(stream tube)演算之觀念，該模式並具備擬似二維水流及擬似三維河床變形之功能。GSTARS模式最早版本是由美國聯邦政府內政部墾務局之資深之工程師 Molinas 及 Yang (1986)所發展，惟此版本僅適用於大型電腦 (mainframe computer)。Yang (1998)等人進一步將GSTARS模式加以改進，升級為GSTARS 2.0版本，並將程式擴展至能應用於個人電腦之操作。近來又擴充部分功能且將程式操作視窗化，升級為GSTARS 2.1版本，目前正進行GSTARS 3.0版本的研發工作。

(二)模式功能

茲將GSTARS 2.0、GSTARS 2.1及GSTARS 3.0版本的發展重點與新增功能整理如表3-2所示，並分述如下。

GSTARS 2.0 模式

1. 將程式更改成適用於個人電腦之操作。
2. 凝聚性沉澱(cohesive sediment)輸運功能。
3. 模式中使用的輸砂公式由 4 個增加至 11 個，包含 10 個非凝聚性沉澱(noncohesive sediment)輸砂公式及 1 組凝聚性沉澱輸砂公式。
4. 以安息角(the angle of repose)為考量之邊坡穩定功能
5. 利用 Han (1980)的遞減函數(the decay function of Han)觀念，模擬非平衡輸砂(non-equilibrium sediment transport)歷程。

(subcritical flow)俱存之混合流況的問題。

5. 動床演算具有如下之功能。

- (1) 具模擬水力篩選(sorting)及甲護(armoring)之功能。
- (2) 具模擬非均勻沉滓之功能。
- (3) 河床質能分層設定不同沉滓粒徑之組成。
- (4) 具模擬非平衡輸砂之功能。
- (5) 具模擬凝聚性沉滓功能。
- (6) 具模擬流管間輸砂沉滓交換的功能。

6. 在渠道側壁處，利用最小功率法(minimum total stream power)作為判斷的準則，可決定渠道幾何變化是在側壁或底床部分。因此，模式具有模擬岸壁沖刷(bank erosion)的功能，並利用安息角觀念考量邊坡穩定之問題。

7. 具模擬支流入流時，水理及輸砂之演算功能。

8. 具水庫演算之功能。

(三)模式限制

1. 模式為擬似二維模式，無法實際模擬變量流的案例。
2. 模式無法模擬流場回流的問題。
3. 水流無法有效反應二次流效應。

(四)綜合評估

GSTARS 3.0 模式相關理論基礎則整理如附錄一所示，模式輸入簡介則如附錄二所示。GSTARS 3.0 模式之綜合評估，可依理論、數值及應用範圍三部探討，如表 3-3 所示。

3.3 NETSTARS 模式

公式直接計算總輸砂量及底床載、懸浮載分開計算後再合併為總輸砂量兩種方法，尤其在高懸浮載的河川或不平衡輸砂現象明顯的區域即需使用後者來計算方能符合實際物理現象。

水理演算中的迴水演算可用來計算亞臨界流、超臨界流或兩者混合的流況（如水躍）。若流管數目選擇 1，則所計算的底床變化成果將類似於 HEC-6 模式。含主支流及局部點側流之河川系統流況及相對應底床變化均可利用 NETSTARS 模擬。節點處理是河川系統模擬的重點，由節點(nodes)連接許多河段（links）可形成河川網路系統，同時也透過節點傳遞各河段的水理及輸砂特性。

NETSTARS 具有 GSTARS 分流管執行輸砂演算的功能。流管數目為使用者在輸入時即選定，在計算過程中流管數目應相同。天然河川泥砂運動種類依其性質可分為推移載(bed load)、懸浮載(suspended load)及沖洗載（wash load），其中推移載及懸浮載合稱河床質載（bed material load）。在河床質載計算(即總輸砂量)方面，程式內有四個公式可供選擇：

(1) Yang's (2) Ackers & White (3) Engelund-Hansen (4) Van Rijn，其中 (2) 及 (3) 適用粒徑範圍為 0.0625 mm-2 mm；(1) 則可用於礫石（Gravel），最大粒徑可為 10 mm；(4) 之適用粒徑範圍為 0.0625 mm-10 mm，計算若超出範圍粒徑時，(1) 超過 150 mm 及小於 0.06 mm 輸砂量以起動條件控制，此區間之計算結果誤差較大，而在 10 mm-150 mm 之間則有適當參數判斷是否應計算輸砂量；(2)則粒徑小於 0.0005 ft 時，所計算之輸砂量誤差也較大，超過 2 mm 也有參數判斷是否應計算輸砂量；(3)及(4)則在粒徑超過 150 mm 及粒徑小於 0.06 mm 的輸砂量均可由起動條件控制，所得結果誤差也較大。推移載計算方面，模式內有三個公式可利用，即 (1) Meyer-Peter and Muller (2) Van Rijn (3)

6. 糙度係數的表示可由資料檔輸入設定，也可用河川流量的函數計算代替，或由半經驗式計算求得，在缺乏資料時可先由公式計算其概略值再行檢定或修正。
7. 若要求精度高而調小 Δt 時，所有演算所使用的邊界條件，程式會在歷線資料內自動做線性內插計算。

(四)綜合評估

NETSTARS 模式功能之綜合評估依理論、數值及應用範圍三部份探討，如表 3-4 所示。

3.4 模式比較分析

根據 GSTARS 3.0 模式(附錄一)、水庫沉滓運移模式(附錄三)與 NETSTARS 模式(附錄四)之理論基礎，可將三個模式依空間維度、水理條件、輸砂條件及其他功能進行評估分析，評估分析結果如表 3-5 所示。茲將綜合評估結果分述如下

一、空間維度：

GSTARS 3.0 模式與 NETSTARS 模式均利用流管的概念，將模擬區域切割成若干個一維河道，使其模擬結果能呈現擬似二維水流及河床變形的功能。水庫沉滓運移模式為水平二維模式，能具體呈現水流及河床在平面空間之變化情形。

二、水理條件：

(1) 亞臨界流與超臨界流

GSTARS 3.0 模式與 NETSTARS 模式均有針對超臨界流流況進行處理，使其對超臨界流及亞臨界流均能進行模擬。水庫沉滓運移模式目前僅能模擬亞臨界流流況。

GSTARS 3.0模式採用河床質載(bed-material load)計算輸砂量；水庫沉滓運移模式將懸浮載與河床載分離演算；NETSTARS模式則上述兩個功能均以建置於模式中。

(2)河床質篩分(bed sorting)與甲護(armoring)

三個模式均針對河床質篩分與甲護進行處理。

(3)凝聚性沉滓

三個模式均能模擬凝聚性沉滓運移的行為。

(4)岸壁沖淤

渠道沖淤現象可能發生在兩個方向，一為底床沖淤而使底床高程降低或升高，一為岸壁沖淤而使渠道變寬或窄。GSTARS 3.0模式利用最小功率法決定河床變形的方向，因此可模擬岸壁沖淤的現象。水庫沉滓運移模式與NETSTARS模式目前沒有針對岸壁沖淤進行處理。

(5)支流入流

三個模式均有處理支流入流的功能。

(6)彎道

在彎道輸砂模擬部分，GSTARS 3.0模式將彎道二次流所導引之較小粒徑沉滓逆側坡而上的傳送納入模式中考量，水庫沉滓運移模式與NETSTARS模式目前對於彎道輸砂並沒有特別的處理。

表 3-1 水庫沉滓運移模式綜合評估表(2/2)

數值部份

結合演算法			分離演算法		*
數值方法		1. 有限差分法 2. 控制體積法			
起始條件		水位高程、底床高程、速度、泥砂粒徑和其特性			
邊界條件		上游	流量		
		下游	水面高程		
格網處理	ΔX	固定		變化	*
	ΔY	固定		變化	*

應用範圍

一維		擬似二維		二維	*
河床質載	*	懸浮載	*	河床載	*
總載	*	黏性泥砂			*
級配	*	篩分	*	護甲	*
均勻質	*	非均勻質			*
超臨界流		亞臨界流			*
直渠道	*	彎道			*
規則渠道	*	不規則渠道			*
單一渠道	*	網路渠道			

表 3-3 GSTARS 3.0 模式綜合評估表(1/2)

理論部分

<p>水 理 方 程 式</p>	<p>1.連續方程式 $Q=AV$ 2.能量方程式 $Y_1 + a_1 \frac{V_1^2}{2g} = Y_2 + a_2 \frac{V_2^2}{2g} + h_f + h_e$</p>
<p>輸 砂 方 程 式</p>	<p>1.輸砂方程式 $\frac{\partial Q_s}{\partial x} + \eta \frac{\partial A_d}{\partial t} + \frac{\partial Q_s}{\partial t} = 0$ 2.輸砂公式 (1)Meyer-Peter and Muller (1948) (2)Laursen (1958) (3)Toffaletti (1969) (4)Engelund and Hansen (1972) (5)Ackers and White (1973) (6)Ackers and White (1990) (7)Yang (1973) +Yang (1984) (8)Yang (1979) +Yang (1984) (9)Parker (1990) (10)Yang et al. (1996) (11)Dubuois (1879) (12)Tsingsua University Equation for Reservoir Flushing (1985) (13)Ashida and Michiue Method (1972)</p>
<p>假 設 條 件</p>	<p>1. 擬似二維定常流 2. 可近似模擬二次流,密度流,床形影響 3. 河岸為可沖刷 4. 斷面可給一條以上之砂石級配曲線</p>
<p>率 定 參 數</p>	<p>1. 河道曼寧糙度係數 2. 流管數目</p>

表 3-4 NETSTARS 模式綜合評估表(1/2)

理論部分

<p>水 理 方 程 式</p>	<p>1. 定量流：連續方程式 能量方程式 2. 變量流：連續方程式 動量方程式</p>
<p>輸 砂 方 程 式</p>	<p>1. 輸砂方程式 $\frac{\partial Q_s}{\partial x} + \eta \frac{\partial A_d}{\partial t} + \frac{\partial Q_s}{\partial t} = 0$ 2. 輸砂公式 河床質載(total load) Yang's formula(bed-material) Ackers and White's equation(bed-material) Engelund-Hanson(bed-material) Van Rijn(bed-material) Meyer-Peter and Muller(bed-load) Van Rijn(bed-load) Schoklitsch(bed-load)</p>
<p>假 設 條 件</p>	<p>1. 擬似二維定常流 2. 忽略二次流, 密度流, 床形影響 3. 河岸為定床不可沖刷 4. 斷面可給一條以上之砂石級配曲線</p>
<p>率 定 參 數</p>	<p>1. 河道曼寧糙度係數 2. 流管數目</p>

表 3-5 模式功能評估比較表

		水庫沉滓 運移模式	GSTARS3.0 模式	NETSTARS 模式
空間維度	擬似二維		●	●
	二維	●		
水理條件	亞臨界流	●	●	●
	超臨界流		●	●
	定量流	●	●	●
	變量流	●		●
	彎道	●		
	回流流場	●		
	支流入流	●	●	●
輸砂條件	河床質載		●	●
	懸浮載與河床載分離	●		●
	河床質篩分甲護	●	●	●
	凝聚性沉滓	●	●	●
	岸壁沖淤		●	
	彎道		●	
	支流入流	●	●	●

第四章 GSTARS 3.0 模式評估測試

針對 GSTARS 3.0 模式之主要功能，美國內政部墾務局提供 6 個相關案例，包括混合流況案例、凝聚性沉澱輸砂案例、流管間沉澱相互傳輸案例、最小功率法應用案例、支流入流案例及水庫演算案例。本計畫利用此 6 個案例進行 GSTARS 3.0 模式之測試工作，茲將各案例之測試成果分述如下。

4.1 混合流況案例

混合流況水理測試採用 MacDonald et al. (1997) 提出具有解析解的水躍案例進行數值演算，此案例為梯形渠道，渠道長度 1000 m，渠底寬度 10 m，梯形渠道邊坡坡度為 1，曼寧係數為 0.02，渠道底床高程變化如圖 4-1 所示。渠道上游入流量為 $20 \text{ m}^3/\text{s}$ ，下游出口水位為 1.63 m。設定流管數為 1(相當於一維演算)，並將河道劃分成 21 個等距計算斷面，數值參數為計算斷面間距 $\Delta s = 50 \text{ m}$ ， $\Delta t = 1 \text{ day}$ ，總模擬時間為 4 days。

水位高程模擬結果如圖 4-1 所示，渠道入游為亞臨界流流況，在距下游出口約 700 m 處流況轉變為超臨界流，至距下游出口 400 m 附近發生水躍，流況再度轉變為亞臨界流流況。圖 4-2 為模式模擬結果與解析解的絕對誤差(定義為 $100\% * (z_{sa} - z_{sm}) / z_{sa}$ ，其中 z_{sa} 為解析解之水位高程； z_{sm} 為模式模擬之水位高程)示意圖，圖中可看出在距下游出口約 400 m 處會發生比較大的誤差(約 20%)，其主要的原由乃在於渠道在此發生水躍，而在該處形成一近似垂直的水面剖線，所以此處發生較大的誤差是可以理解的。整體而言，模式模擬結果與解析解的差異不大，驗證 GSTARS 3.0 模式模擬混合流況的正確性。

4.2 凝聚性沉澱輸砂案例

因砂行為。表 4-1 為入庫沉滓流達率一覽表，由表中可看出細砂沉滓的流達率均為 100%，而凝聚性沉滓隨著粒徑變小，流達率有降低的趨勢，此乃因為粒徑愈小的凝聚性沉滓其比面積愈大，沉滓顆粒間的凝聚結合效應愈強，沉降的機會愈大。整體而言，入庫沉滓的因砂現象並不顯著，不同沉滓的流達率約可達 9 成以上，各沉滓的總流達率更高達 98.24%。

4.3 流管間沉滓相互傳輸案例

根據 GSTARS 3.0 模式之理論架構，側向流管間沉滓之相互傳輸機制主要有兩個(如圖 A-5 所示)，一為因重力效應所導引之較大粒徑沉滓順側坡而下的滾動，一為因彎道二次流所導引之較小粒徑沉滓逆側坡而上的傳送。設定三個案例以測試模式沉滓側向傳輸之機制，案例一忽略沉滓側向相互傳輸之機制，案例二設定沉滓側向相互傳輸僅受重力效應之影響，案例三設定為沉滓側向相互傳輸同時受重力與二次流效應之影響。

模擬渠道長度 338.7 m，渠道寬度 12.2 m，渠底側向坡度為 14° (如圖 4-6 所示)，渠道曲率半徑為 176.8 m，曼寧係數為 0.02。渠道上游入流量為 $9.91 \text{ m}^3/\text{s}$ ，上游入砂量設定為平衡輸砂情況(根據水流之挾砂能力計算入砂量)，下游出口水位設定為正常水深。設定流管數為 5(其中流管編號愈大者，其初始底床高程愈低)，並將河道劃分成 7 個計算斷面，輸砂模擬採用非均勻質演算，3 個不同沉滓粒徑區間分別為 0.02~0.5 mm、0.5~2 mm 及 2~12.5 mm。輸砂演算採用 Yang (1973) 之輸砂公式，數值參數 $\Delta t = 20 \text{ min}$ ，總模擬時間為 1000 hr。

圖 4-7 為模擬時間終了時三個案例流管總輸砂量沿程變化圖，其中 (a)~(e) 分別為流管編號 1~5 之模擬結果。案例一忽略流管間相互傳輸之機制，在平衡輸砂的情況下，由圖 4-7 可看出各流管之總輸砂量都維持在穩定的狀態。案例二考量重力效應的影響，所以流管間沉滓的相互傳輸為底床

mm、5~10 mm 及 10~20 mm。設定流管數為 3，並將河道劃分成 23 個計算斷面，輸砂演算採用 Yang's sand (1973)與 gravel (1984)輸砂經驗公式， $\Delta t = 20 \text{ min}$ ，總模擬時間為 2000 min。

為呈現最小功率法之功能，分別採用忽略與考量最小功率法的案例進行模擬。圖 4-10 為兩者之模擬結果與原始床形沿河心距方向之比對圖，由圖中可看出整個模擬河道均呈現沖刷的情況，且忽略最小功率法之底床刷深量會普遍大於考量最小功率法之刷深量。此結果之主要原因乃在於忽略最小功率法之模擬時，河道沖刷僅能反應在底床部份，而不能反應在側壁方向，所以其刷深量會普遍大於考量最小功率法之模擬結果。此外，在距上游溢洪道出口約 18.3 m 處(圖 4-9 中 0+60 處)有斷面量測資料，圖 4-11 為忽略與考量最小功率法之模擬結果與實測資料之比對圖，由圖中可看出忽略最小功率法之模擬會產生局部底床過度刷深的不合理情況，且其主深槽亦與量測值相異。而考量最小功率法之模擬結果能同時反應河道斷面刷深與變寬的情況，模擬結果與實測資料比對呈現合理之趨勢，驗證 GSTARS 3.0 模式最小功率法之實用性。

4.5 支流入流案例

假定支流入流案例其主流渠道總長度為 1,500 m，在距下游出口 914 m 處有一支流匯入主流。此主流渠道為一梯形渠道，渠底寬度為 61 m，渠道邊坡坡度為 1:2 (垂直:水平)，縱向坡度為 0.0001，曼寧係數為 0.03。主流渠道上游入流量及入砂量分別為 $85 \text{ m}^3/\text{s}$ 及 86.28 公噸/天，渠道下游出口水位設定為正常水深(normal depth)。分別設定兩個支流入流案例進行模擬，案例一為淤積案例，在模擬時間 0 至 500 hr 時支流入流量及入砂量均為零，在模擬時間 500 至 1500 hr 時支流入流量及入砂量分別為 $5.7 \text{ m}^3/\text{s}$ 及 108.9 公噸/天；案例二為沖刷案例，在模擬時間 0 至 500 hr 時支流入流量及入砂

如圖 4-14 所示，河道曼寧係數為 0.035。設定流管數為 1，並將模擬河段劃分成 17 個計算斷面，水庫初始水位高程為 67.06 m，數值參數 Δs 約為 800 m， $\Delta t=1$ day，總模擬時間為 10 days。

圖 4-15 為不同時刻水位高程模擬結果，對照圖 4-14，在模擬時間第 1 天水庫入流量大於出流量，第 2 天水庫入流量等於出流量，所以在模擬時間第 2 天終了時，水庫蓄水量會增加，水庫水位應呈現上升的情況。由圖 4-15 可看出在模擬時間第 2 天終了時($t=2$ days)，水庫水位約為 69.2 m，大於模擬初始水位高程，呈現合理的蓄水增加情況。在模擬時間第 3 至 4 天時，水庫出水量持續增加，且出水量會大於入流量，此時水庫蓄水量會減少，水庫水位應呈現下降的情況。由圖 4-15 可看出在模擬時間第 4 天終了時($t=4$ days)，水庫水位降至約 66.3 m，呈現合理的蓄水減少情況。在模擬時間第 5 至 7 天時，水庫出水量持續減少，但此時出水量仍大於入流量，所以水庫蓄水量及水庫位仍會持續減少與降低。由圖 4-15 可看出在模擬時間第 7 天終了時($t=7$ days)，壩址處水位降至約 58.22 m。在模擬時間第 9 至 10 天時，水庫入流量又呈現大於出流量的情況，由圖 4-15 可看出在模擬時間第 10 天終了時($t=10$ days)，水庫蓄水量增加，水位上升至約 65.9 m。綜合上述之模擬分析，GSTARS 3.0 模式能合理模擬水庫蓄水變化所造成的水庫水位變化歷程，驗證該模式水庫演算之合理性。

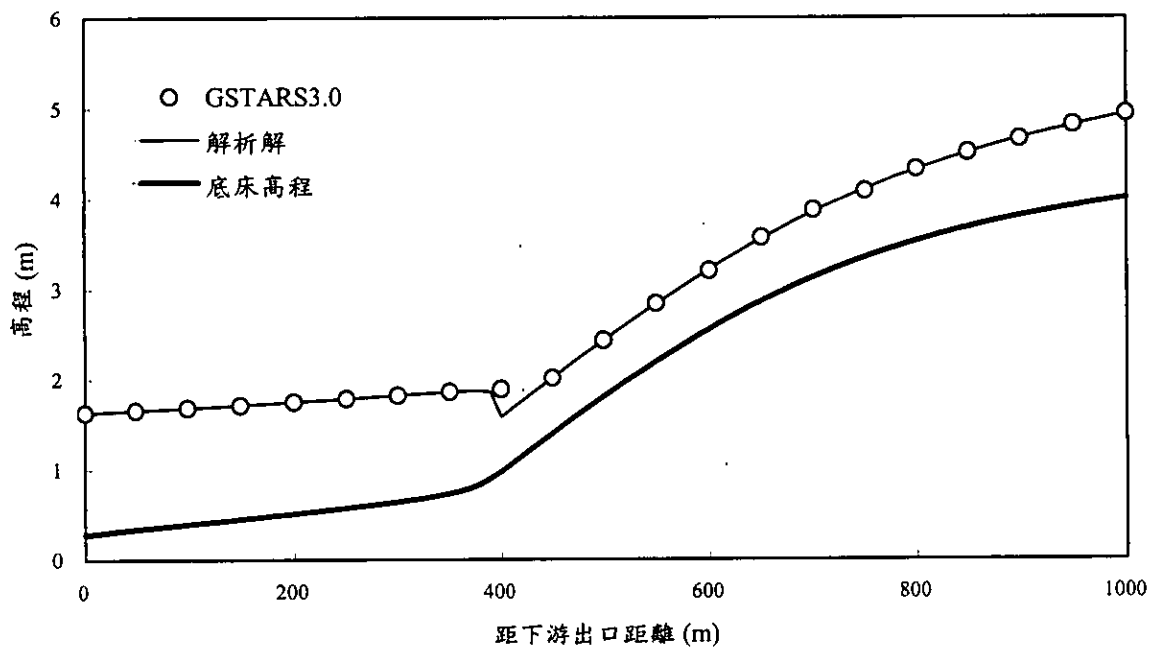


圖 4-1 混合流況案例水位高程模擬結果

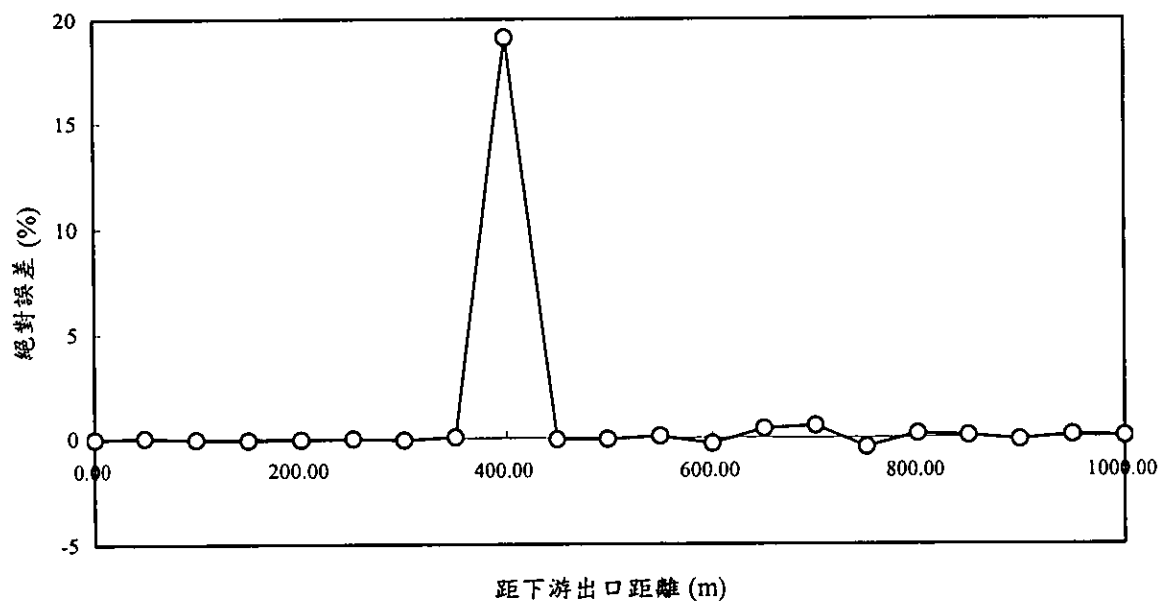


圖 4-2 混合流況案例模擬結果之絕對誤差

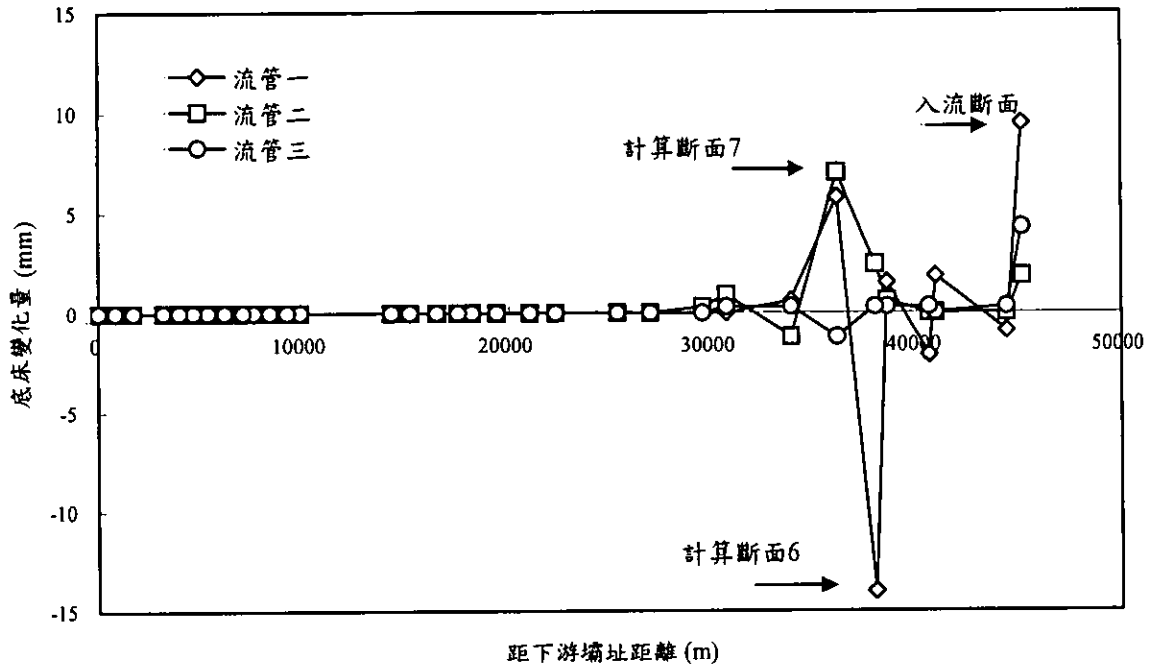


圖 4-5 凝聚性沉滓輸砂案例底床高程模擬結果

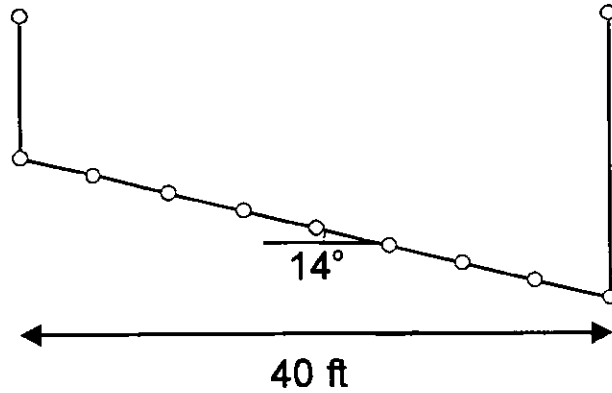
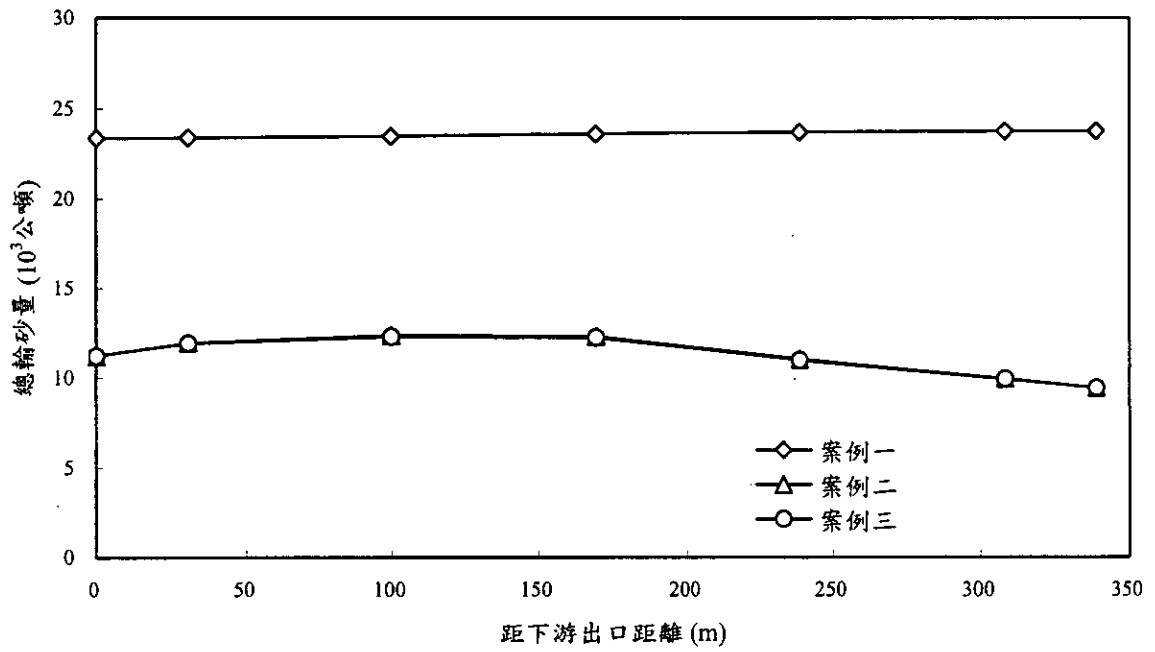
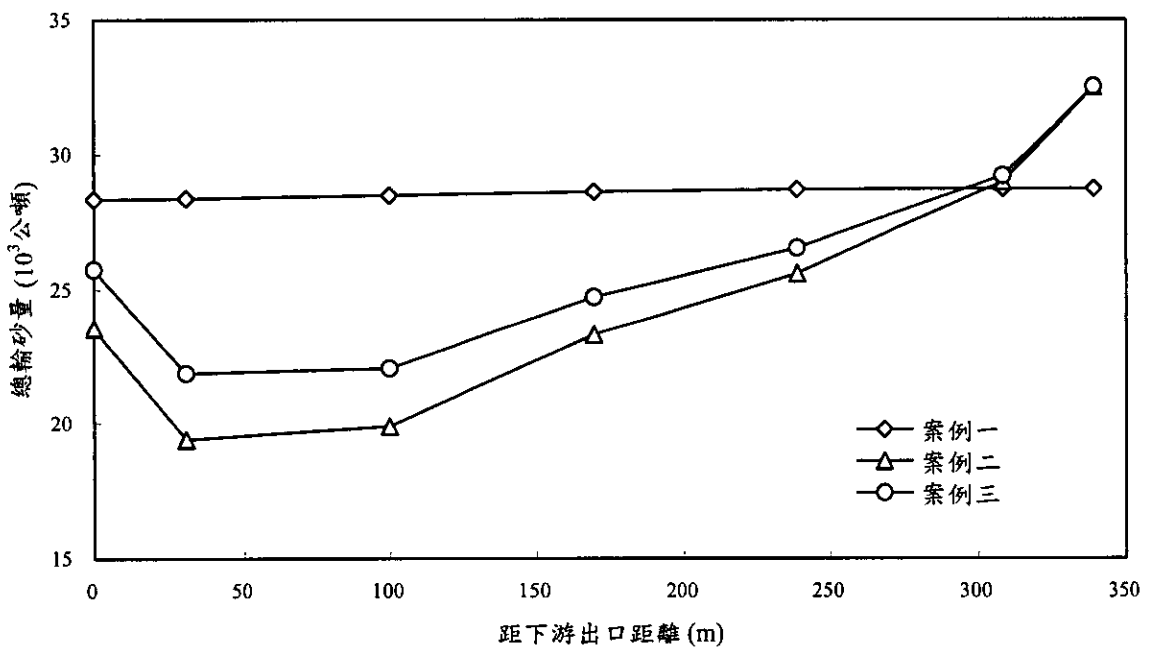


圖 4-6 流管間沉滓之相互傳輸案例河道断面示意图



(c)



(d)

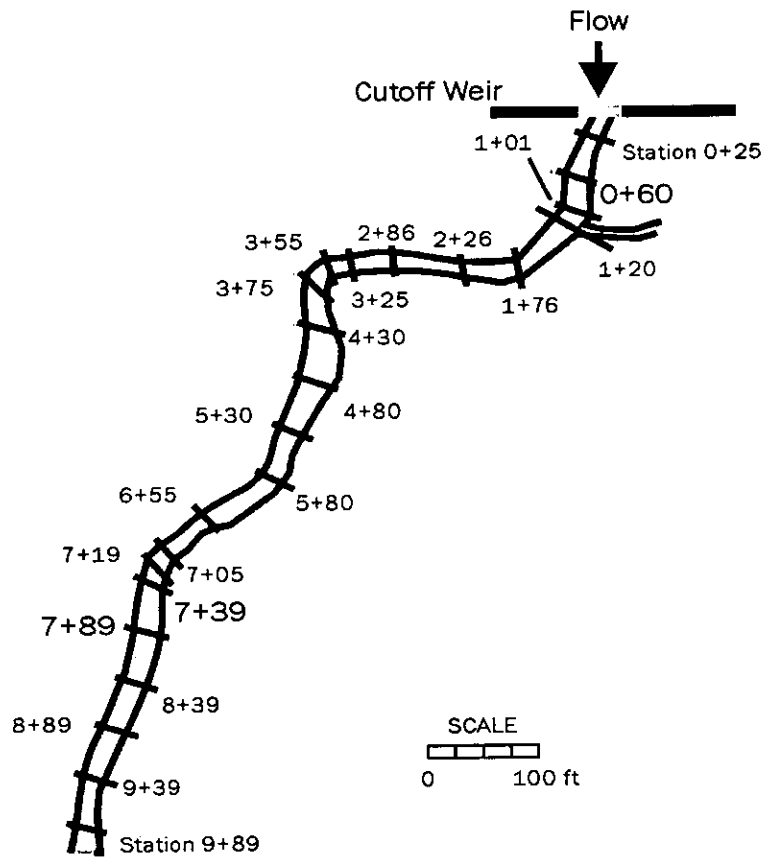


圖 4-9 最小功率法應用案例幾何示意圖

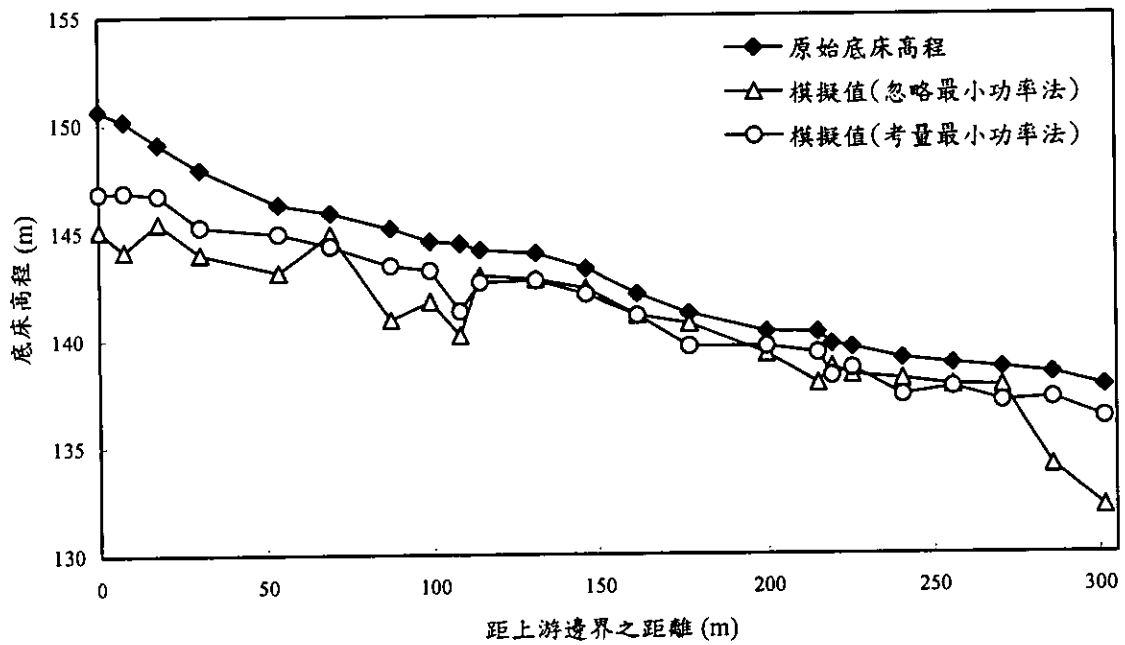


圖 4-10 忽略與考量最小功率法案例底床高程模擬結果比對圖

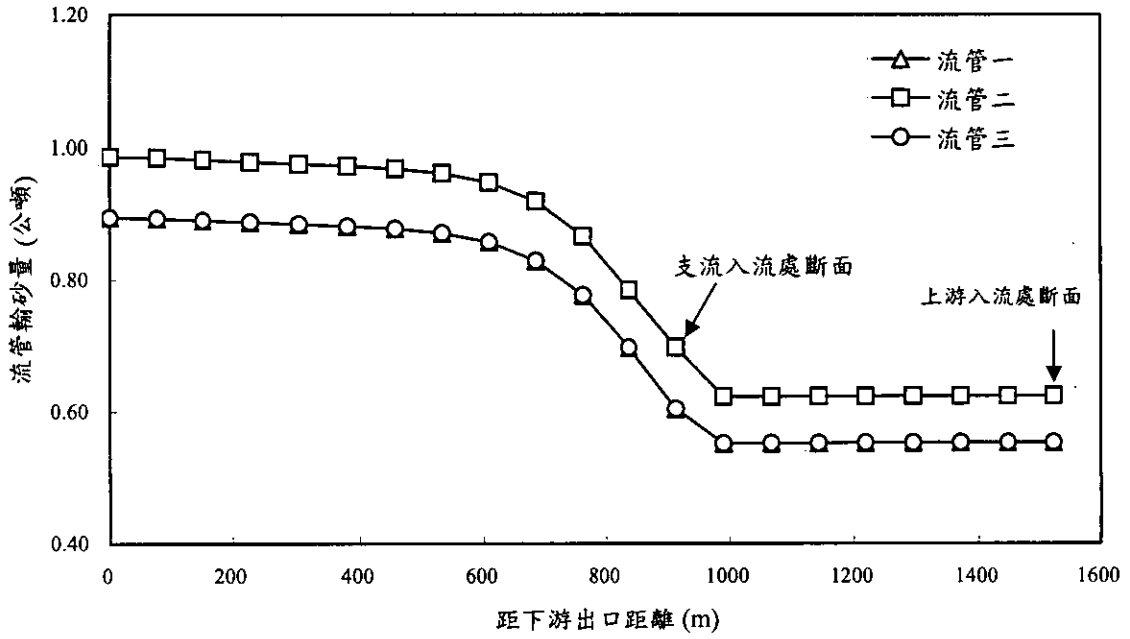


圖 4-13 案例二各流管總輸砂量模擬結果

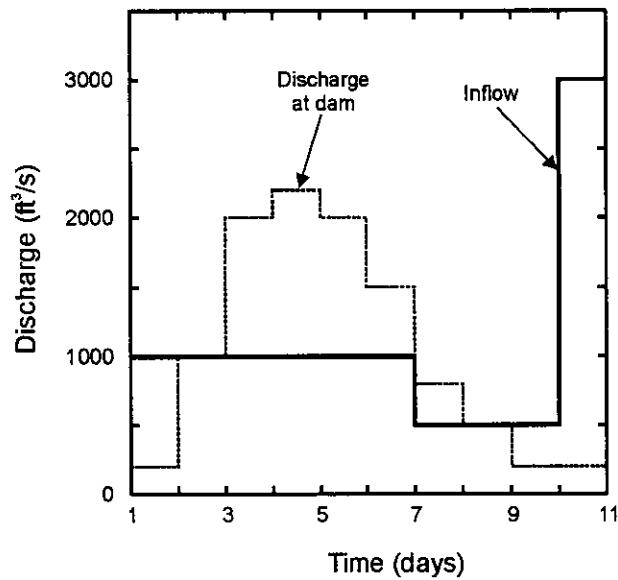


圖 4-14 水庫演算案例上游入流量及下游壩址處出流量示意圖

第五章 水庫沉滓運移模式之發展與測試

5.1 模式發展概述

在模式功能部份，針對石門水庫浚渫評估方案之模擬，模式應具備能模擬下列流況的功能：

1. 變量流：以合理模擬洪水歷線之漲退水過程。
2. 彎道：在霞雲雨量站附近為一明顯蜿蜒渠道，因此，模式應具有彎道模擬的能力。
3. 回流流場：以反映水庫蓄水效應與幾何不規則變化。
4. 非平衡輸砂：為反水庫非平衡輸砂的現象，模式發展時採用懸浮載與河床載分離的演算方式。
5. 凝聚性沉滓：為具體反映水庫泥砂的特性，模式應包含模擬凝聚性沉滓的功能，包含沖刷、沉積與底床壓密(consolidation)的功能。

在理論基礎部份，透過座標系統轉換將控制方程式轉換為正交曲線座標(orthogonal curvilinear coordiante)系統方程式，並利用自由水面、底床之運動邊界條件及輔助關係式將控制方程作時間平均(time-average)及水深平均(depth-average)，則可推得水平二維正交曲線座標控制方程式。其中水理控制方程包含水理連續及動量方程式；輸砂部份將輸砂通量分離為非均勻質之懸浮載與河床載，所以其控制方成包含某一粒徑之懸浮載質量守恆方程式、某一粒徑於作用層(active layer)之質量守恆方程式及整體河床輸砂之質量守恆方程式。水庫沉滓運移模式之基本理論基礎，包含水理及輸砂之控制方程式、輔助關係式及數值處理部份之詳細內容請參閱附錄三。

5.2 水理模式測試驗證

分析彎曲渠道之水理分佈是很複雜的問題，在最近幾十年來有相當多的研究利用實驗和數值方法來探討彎道的問題(Rozovskii 1961；de Vriend and Koch 1977；Yeh and Kennedy 1993)。本研究採用 de Vriend and Koch (1977)之實驗數據，驗證本模式模擬彎道的能力。

實驗水槽佈置如圖 5-5 所示，從斷面 A_0 至斷面 B_0 為直線道，斷面 B_0 為彎道之起點，沿著渠道一直到斷面 E_0 為一蜿蜒 90° 之固定曲率半徑之定床矩形渠道。渠道長 38 m，渠寬 6 m，渠道中心線曲率半徑為 50 m，渠道縱向坡度在直線段維持水平，在彎道內為 0.0003，Chezy 係數為 $70 \text{ m}^{1/2}/\text{s}$ 。上游邊界條件為固定入流量 $0.61 \text{ m}^3/\text{s}$ ，下游邊界條件為實測水位資料。採用 103×39 的非均勻計算格網， $\Delta t = 2.5 \text{ s}$ 。

圖 5-6 為流速模擬結果與實驗值之比較圖，圖中縱軸之無因次參數為比較點之水深平均流速 U 除以該斷面的平均流速 U_M ，橫軸之無因次參數為比較點曲率半徑 R 減去渠道內岸之曲率半徑 R_I 後再除以渠道寬度 B 。圖 5-7 為水位模擬結果與實驗值之比較圖，圖中縱軸為比較點水位高程 Z_c 減去該斷面平均水位高程 Z_{sm} 除以水深 d 後，再乘上一百倍之放大尺度無因次參數，橫軸之定義如同流速比較圖。由圖 5-6 及 5-7 可看出流速、水位模擬結果與實驗數據相當吻合。如圖 5-6 所示，流場受到二次流效應的影響，呈現外岸流場加速的現象，此彎道中下游段受此效應的影響，外岸縱向流速會明顯大於內岸的縱向流速。如圖 5-7 所示，受彎道效應的影響，斷面水面高程會呈現所謂水面超高(superelevation water surface)的分佈，即外岸水面高程會大於內岸的水面高程。此水面超高的現象甚至從彎道入口處(斷面 B_0)即已發生，至斷面 B_1 時，此斷面水面高程的側向坡度即已達穩定的狀態。

道下半部會產生環流流場的合理流況。

5.3 輸砂模式測試驗證

5.3.1 沖刷案例

渠道內因入流砂量小於水流之挾砂能力所引起的底床下降過程稱之為沖刷，本計畫採用 Suryanarayana (1969)的實驗案例，驗證輸砂模式之沖刷機制。實驗渠道為一直線矩形試驗水槽，長 18.3 m、寬 0.6 m，渠道上游處裝置一加砂器，下游末端裝置一沉滓收集器。本文中引用的案例，所用之沉滓包含兩種標示為粒徑二與粒徑三的組成，其粒徑分佈曲線如圖 5-10 所示，其中粒徑二可視為均勻沉滓，粒徑三則為非均勻沉滓。實驗過程中，渠道沿程每隔 0.305 m 紀錄底床高程資料，每隔 1.525 m 紀錄水面高程資料。

5.3.1.1 均勻沉滓案例

均勻沉滓沖刷案例上游邊界為清水入流，單位寬度入流量為 $0.0236 \text{ m}^2/\text{s}$ ，下游邊界水位高程為 0.251 m，底床沉滓採用粒徑二，在模擬案例中以其相對之中值粒徑 0.45 mm 為代表粒徑，孔隙率為 0.3。數值模擬的過程，由啟始平衡條件開始($t=0$)，於 1.5、4 和 9 hr 分別紀錄渠道沿程之底床高程與水位高程，並與實驗量測資料相比較，以驗證模式對於渠道沖刷過程的預測能力。

圖 5-11 為底床高程與水位高程模擬結果與實驗資料之比較圖，由圖中可看出在 $t=1.5 \text{ hr}$ 時，明顯的底床沖刷僅發生在靠上游的渠段，下游段雖有沖刷發生，唯其影響被上游渠段沖刷而起的砂源所抵銷，因此下游段幾乎仍維持在初始的平衡狀態。隨著時間的增加，沖刷現象漸漸的向下游傳遞，至 $t=9 \text{ hr}$ 時，整個模擬渠道底床均呈現沖刷的情況，造成渠道底床

淤積機制。此淤積案例上游邊界單位寬度入流量為 $0.0236 \text{ m}^2/\text{s}$ ，下游邊界水位高程為 0.2565 m 。上游入砂與底床沉滓均採用粒徑二(即均勻沉滓)，代表粒徑為 0.45 mm ，上游入流沉滓體積濃度為 409 ppm ，孔隙率為 0.4 。

圖 5-13 為模擬時間 1.5 、 2.5 、 4.5 、 7 、 10 hr 時，水位與底床高程模擬結果與實驗資料之比較圖，由圖中可看出模擬結果與實驗資料相當吻合，驗證模式模擬渠道淤積歷程的能力。由圖 5-13 中可觀察出此案例的淤積過程，在 $t=1.5 \text{ hr}$ 時上游端附近即會出現明顯的淤積波前，且隨著試驗時間的增加，此淤積波前會如同湧浪般的向下游傳遞。在淤積波前的上游，底床高程會因沉滓落淤而上升，但淤積速率會因水深變淺，底床坡度變緩，致使水流挾砂能力增加而逐漸減緩，最後達到新的平衡狀態，淤積現象近乎停止。另一方面，在淤積波前尚未傳達的下游渠道，則維持原有之平衡狀態，直到淤積波通過，淤積現象方才在該渠道發生。

5.3.3 沖淤交替案例

除了上述個別的沖刷與淤積現象之外，天然河道中存在著更為複雜的沖淤交替現象。本計畫採用台大水工試驗所顏等(1988)所進行的物理模型試驗，驗證本模式對於非均勻沉渠道沖淤交替的模擬能力。

試驗水槽長 72 m ，寬 1 m ，渠道初始底床坡度為 0.0035 ，上游邊界入流量維持固定流量 0.12 cms 。根據試驗資料，採用 5 個代表粒徑來表示上游入砂與底床沉滓之組成，分別為 0.42 、 2 、 4.2 、 6.2 和 11 mm ，各代表粒徑佔入砂濃度與河床質組成的比例則列如表 5-2 所示。此沖淤交替案例共分為四個階段進行，依次為 $0\sim 1175 \text{ min}$ 時，上游以 3.3 kg/min 定量沉滓供應以進行平衡試驗； $1176\sim 2570 \text{ min}$ 時，提高沉滓供應量至 9.9 kg/min 進行超載試驗； $2571\sim 4795 \text{ min}$ 時，恢復沉滓供應至 3.3 kg/min 進行減載試驗； $4796\sim 5100 \text{ min}$ 時，停止上游沉滓供應量進行清水沖刷試驗。

表 5-1 無因次環流長度 L/d_e 及無因次回流量 q_m/U_0d_e 實驗及模擬

結果一覽表

實驗值		模擬結果			
		水庫沉滓運移模式		CV 模式	
L/d_e	q/U_0d_e	L/d_e	q/U_0d_e	L/d_e	q/U_0d_e
7.87	0.085	7.68	0.082	7.43	0.109

表 5-2 沖淤交替案例入砂條件與初始床質粒徑組成

代表粒徑 (mm)	入砂體積濃度 (ppm)	初始床質粒徑組成 (%)
0.42	19.92	36.00
2.00	22.01	37.00
4.20	3.67	10.00
6.20	3.14	8.00
11.00	3.67	9.00

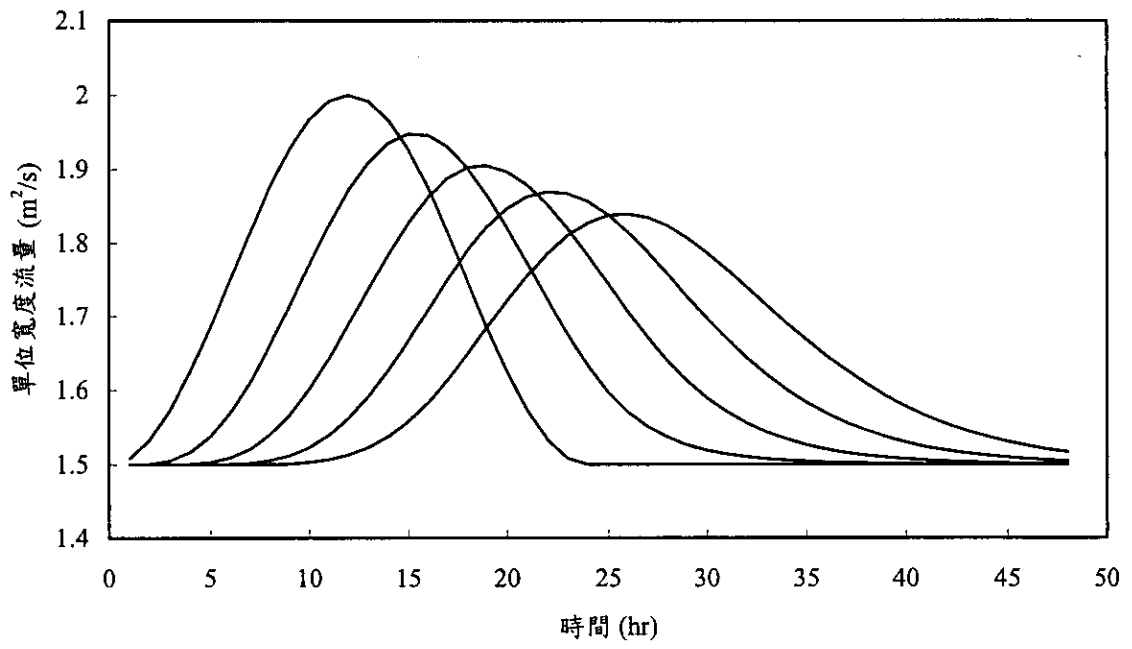


圖 5-3 變量流案例單位寬度流量模擬結果示意圖

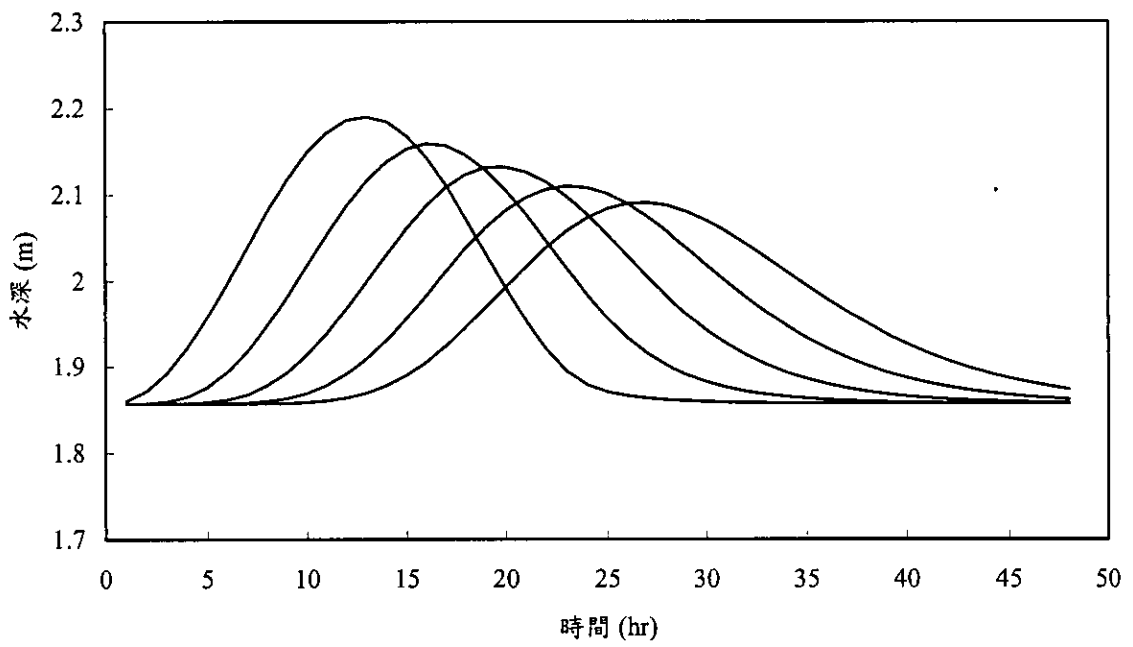


圖 5-4 變量流案例水深模擬結果示意圖

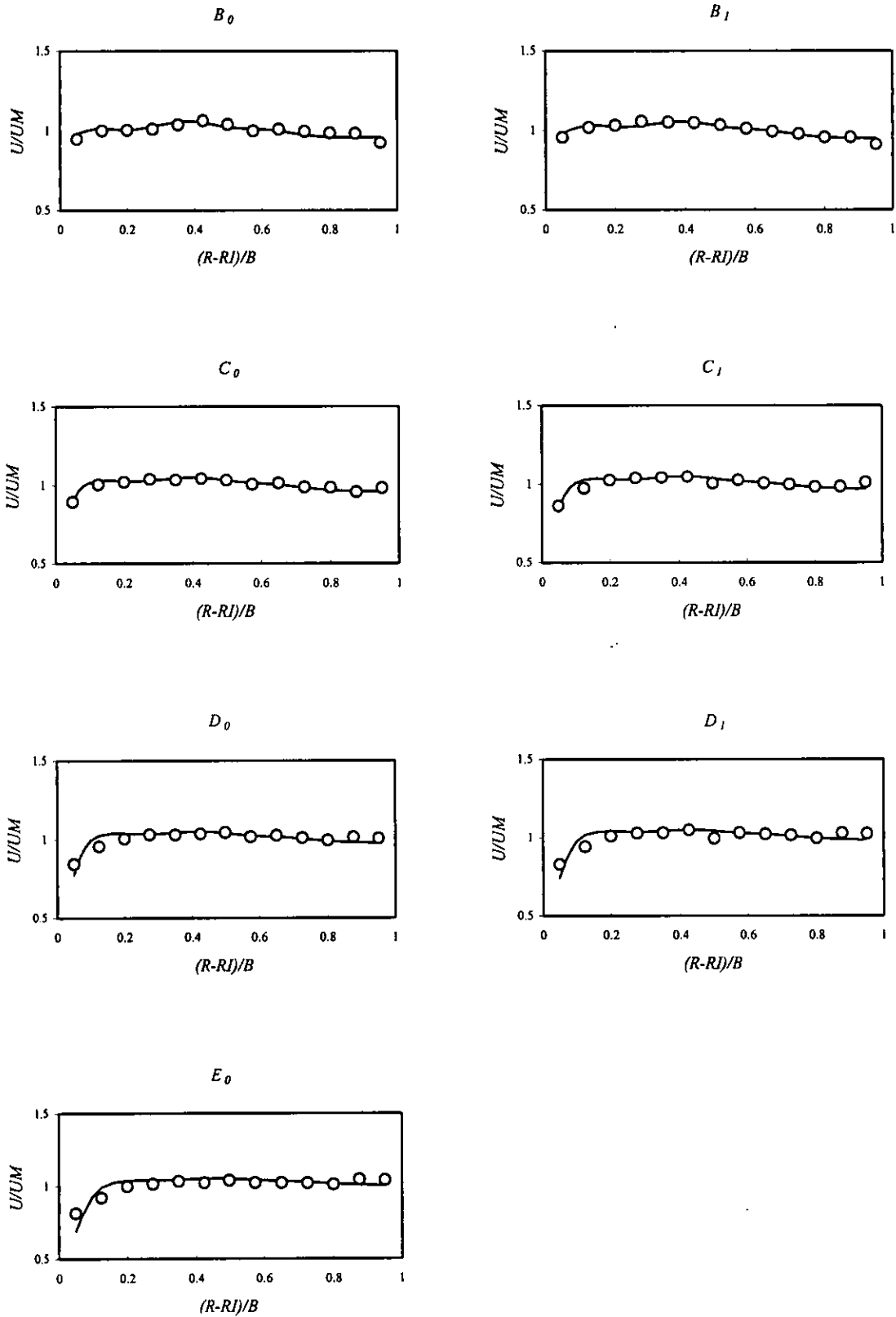


圖 5-6 無因次參數 U/UM 在側方向之比較圖。實驗值(○)；計算值(—)

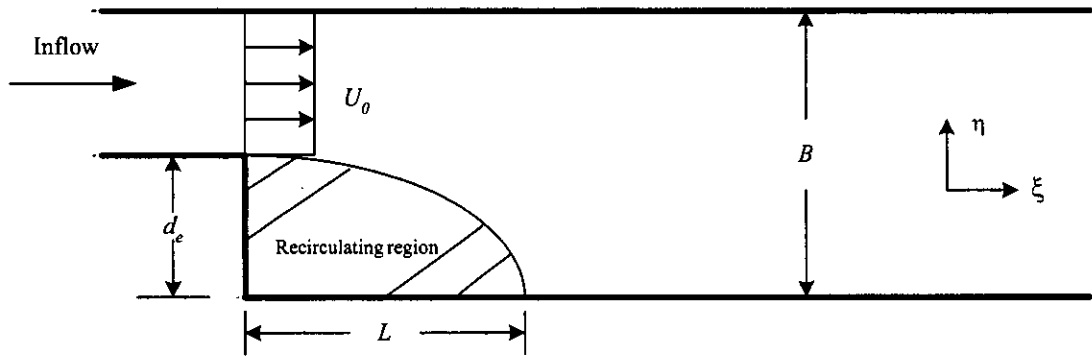


圖5-8 Babarutsi et al. (1989)實驗水槽佈置圖

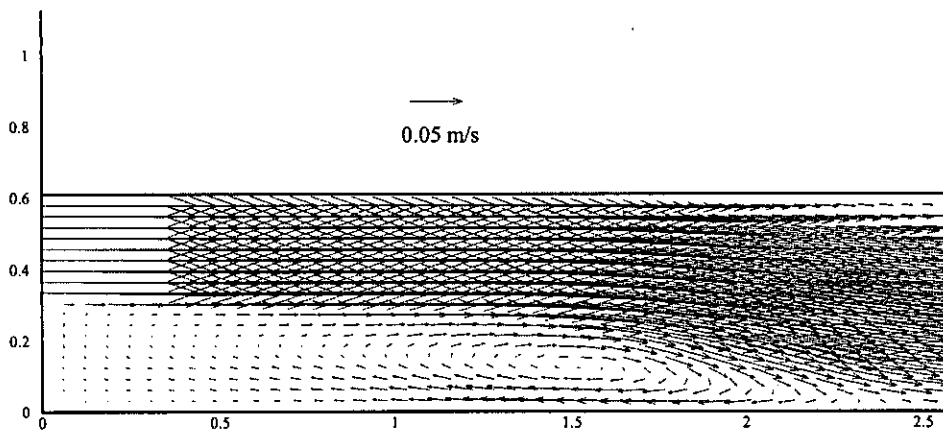
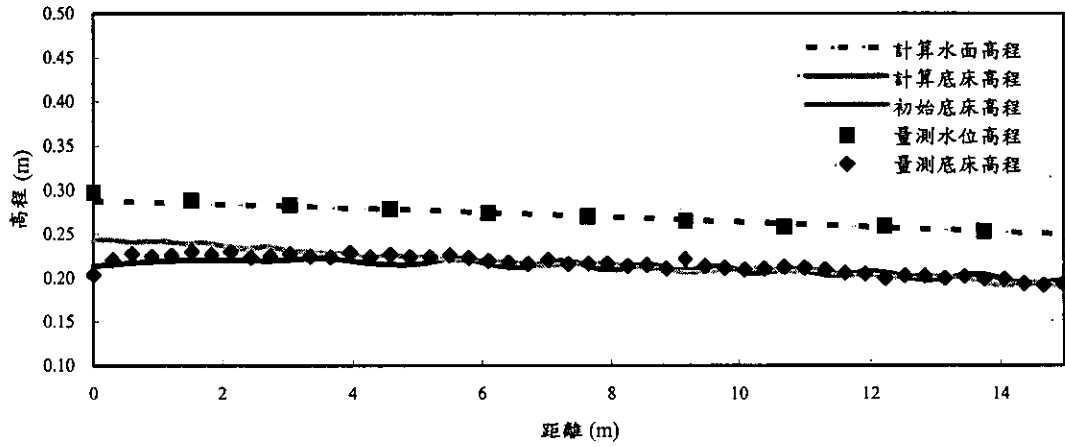
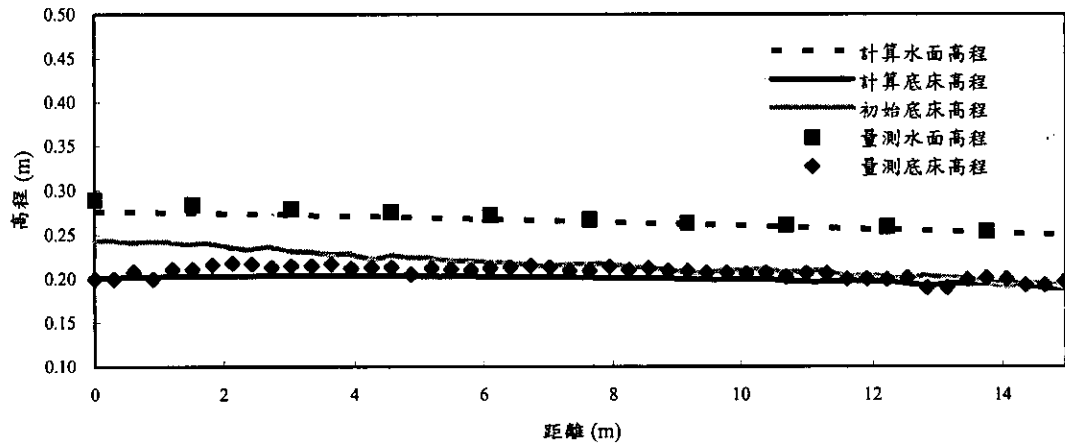


圖 5-9 環流流場分佈圖

1.5 hr



4 hr



9 hr

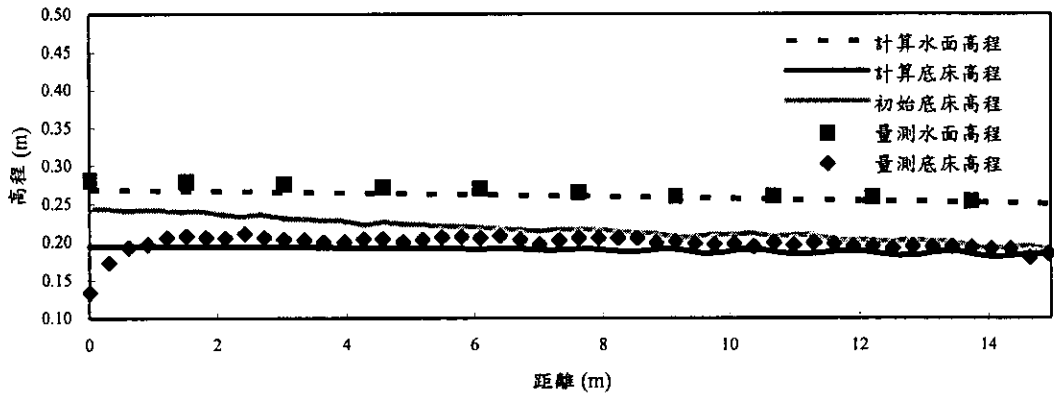


圖 5-11 均勻沉澱沖刷案例之底床沖刷時變圖

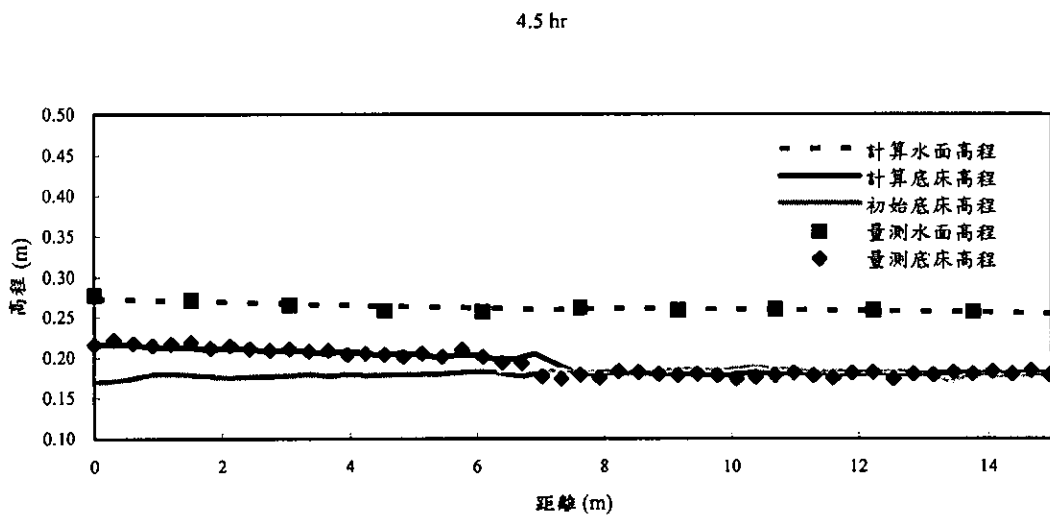
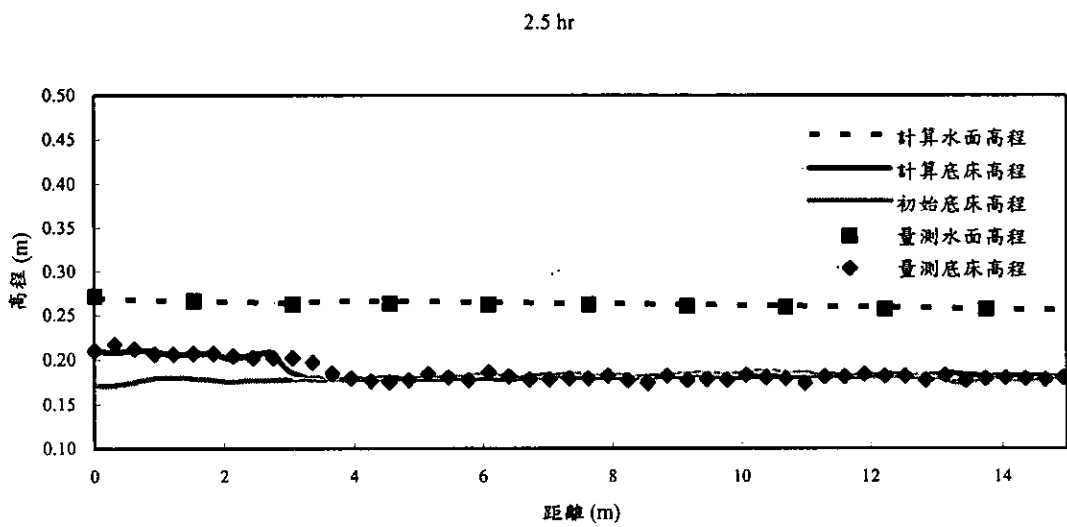
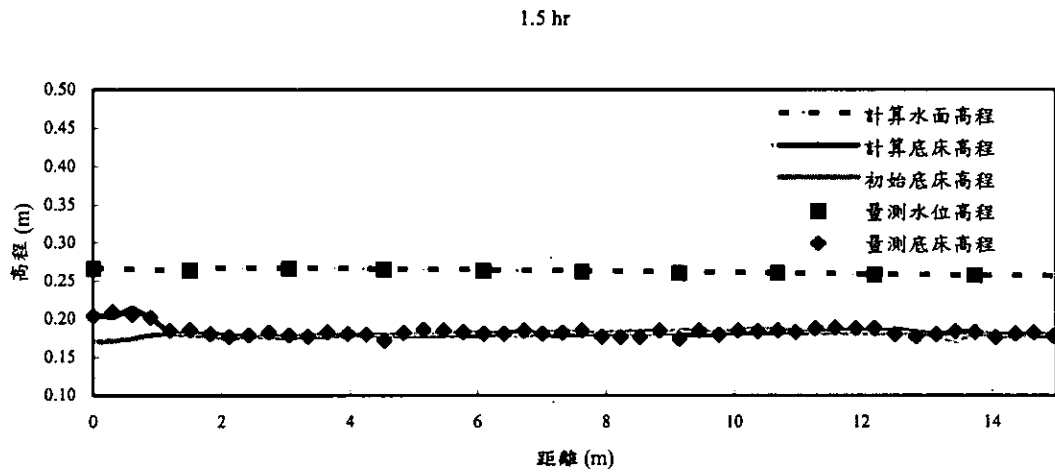


圖 5-13 淤積案例之底床沖刷時變圖(1)

19.58 hr

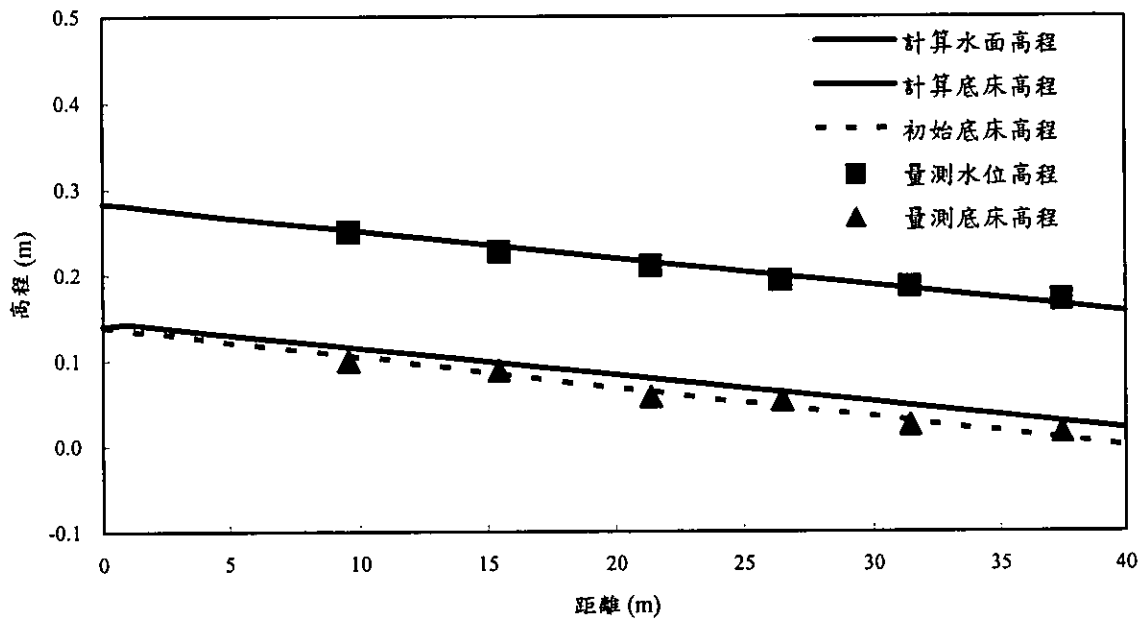


圖 5-14(a) 沖淤交替案例平衡試驗後之底床與水位高程縱向剖面圖

42.83 hr

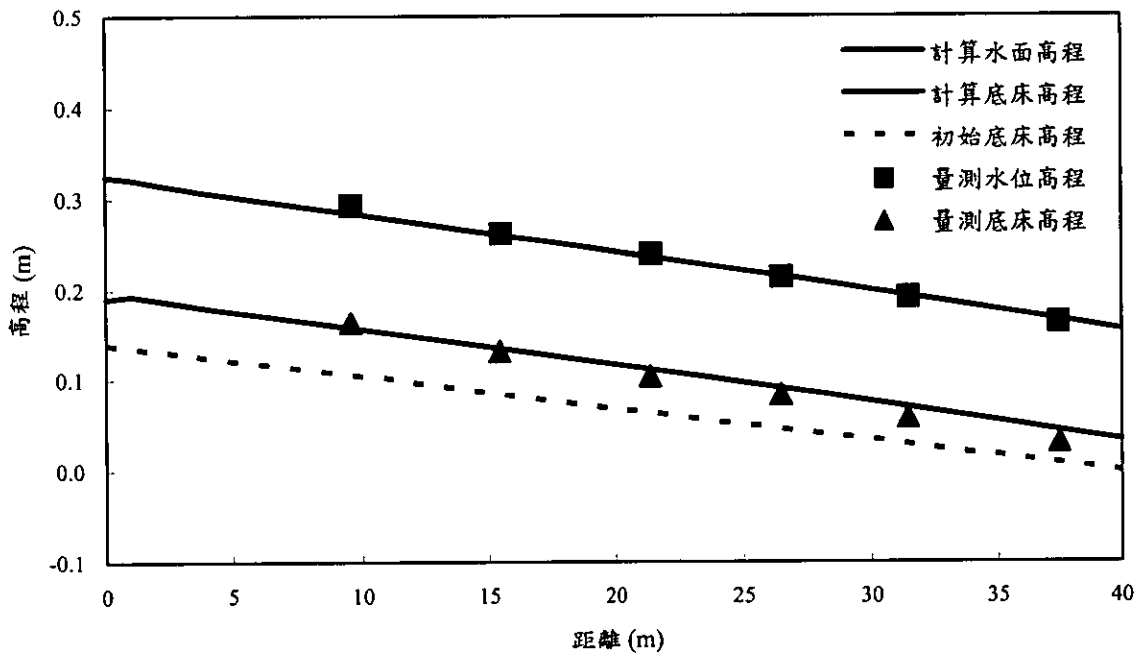


圖 5-14(b) 沖淤交替案例超載試驗後之底床與水位高程縱向剖面圖

第六章 水庫沉滓運移模式與GSTARS 3.0模式比較分析

針對石門水庫應用例之需求，本章節選用數個水理及輸砂案例，分別利用GSTARS 3.0模式與水庫沉滓運移模式進行案例模擬，水理部份包括迴水演算案例、變量流案例、彎道案例及環流流場案例，輸砂部份包括沖刷與淤積案例。藉由比較分析兩模式之模擬結果，可初步評估擬似二維模式二維模式應用於石門水庫之適用性。

6.1 迴水演算案例

案例模擬條件如 5.2.1 節所述。水庫沉滓運移模式採用 81×11 的均勻計算格網，數值參數為 $\Delta x = 100$ m， $\Delta y = 10$ m， $\Delta t = 100$ s。GSTARS 3.0 模式採用 41 個等距計算斷面，數值參數為 $\Delta x = 200$ m， $\Delta t = 1$ day，流管數為 1。

圖 6-1、6-2 分別為兩模式水深及流速模擬結果比對圖，由圖中可看出兩模式模擬結果除了在渠道上游端有些許差異外，兩模式之模擬結果趨於一致，均能有效反應迴水效應的影響。即在流場僅為一個方向流動的案例中，擬似二維與二維模式均能得到合理的結果。

6.2 變量流案例

案例模擬條件如 5.2.2 節所述。水庫沉滓運移模式採用 101×9 的均勻計算格網，數值參數為 $\Delta x = 1000$ m， $\Delta y = 12.5$ m， $\Delta t = 240$ s。GSTARS 3.0 模式採用 101 個等距計算斷面，數值參數為 $\Delta x = 1000$ m， $\Delta t = 240$ s，流管數為 1。

為觀測流量、水深隨空間及時間變化之情形，在沿渠道主流方向每隔 20 km 設一參考點。圖 6-3、6-4 分別為兩模式單位寬度流量及水深模擬結

流場分佈圖，由圖中可看出 GSTARS 3.0 模式無法模擬出環流流場的流況。GSTARS 3.0 模式雖然經由流管的切割後，能展示擬似二維的水流，但各流管基本上仍是一維演算，水流質量與動量的交換僅能反映在縱方向，無法模擬水流在側方向的交換機制。因此，GSTARS 3.0 模式無法如水庫沉滓運移模式般，合理模擬出水流同時具縱向與側向傳輸的環流流場案例。

6.5 沖刷與淤積案例

沖刷與淤積模擬案與 5.3.1.1 與 5.3.2 節相同。水庫沉滓運移模式採用 51×5 的均勻計算格網， $\Delta t = 1$ s。GSTARS 3.0 模式採用 51 個均勻計算格點，數值參數為 $\Delta x = 0.3$ m， $\Delta t = 1$ min，流管數為 1。

水庫沉滓運移模式之模擬結果如圖 5-11 及 5-13 所示，由圖中可看出該模式能有效模擬底床沖淤之變化歷程。GSTARS 3.0 模式之模擬結果如圖 6-8 及 6-9 所示，由圖中可看出當使用 GSTARS 3.0 模式進行模擬時，沖刷及淤積案例的模擬結果均顯示底床變量幾乎為零，每個時刻的底床高程均保持與初始底床高程相同，即 GSTARS 3.0 模式無法合宜模擬底床之沖淤歷程。值得注意的是，在淤積案例中(如圖 6-8 所示)，隨著時間的增加，渠道上游段水面高程有逐漸升高的趨勢，在上游固定流量與下游幾乎固定水位的邊界設定情況下，理論上此水面升高的現象應是床形改變的影響，但其輸出資料卻顯示床形變化量卻為零。為進一步確定問題之所在，本計畫執行團隊已將此模擬案例的資料(包含模式輸入檔)寄給 GSTARS 3.0 模式之研發單位。若有更新結果，將於未來報告補列。

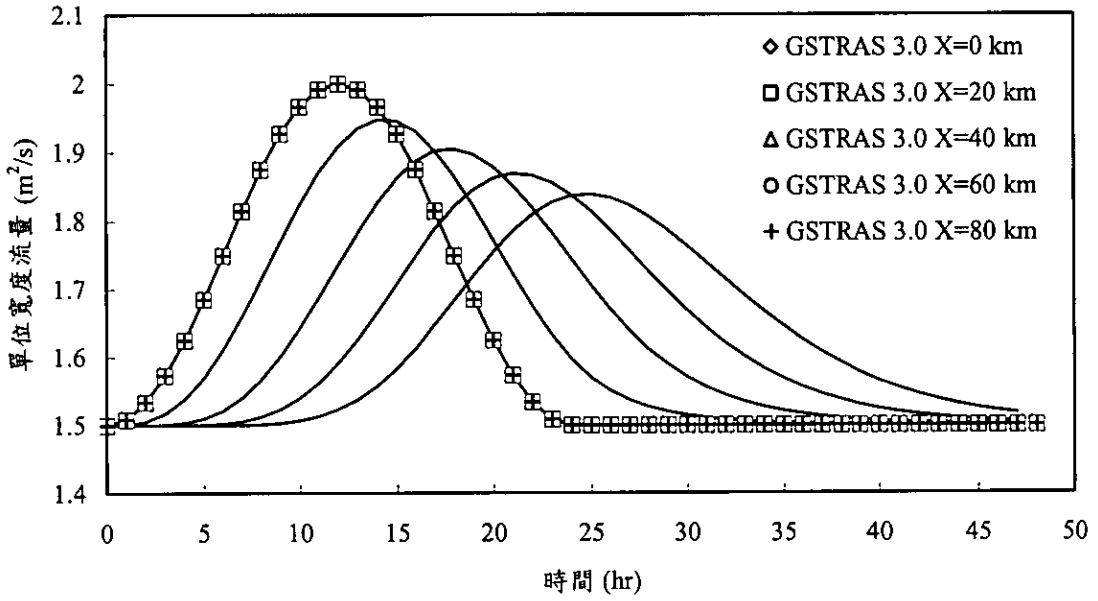


圖 6-3 變量流案例單位寬度流量模擬結果比較圖

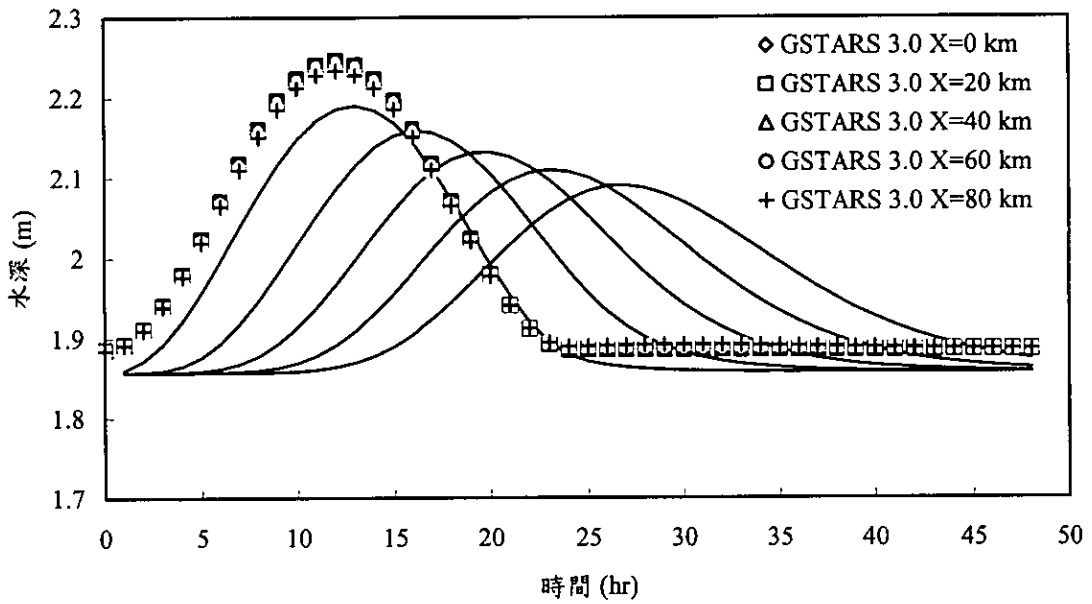


圖 6-4 變量流案例水深模擬結果比較圖

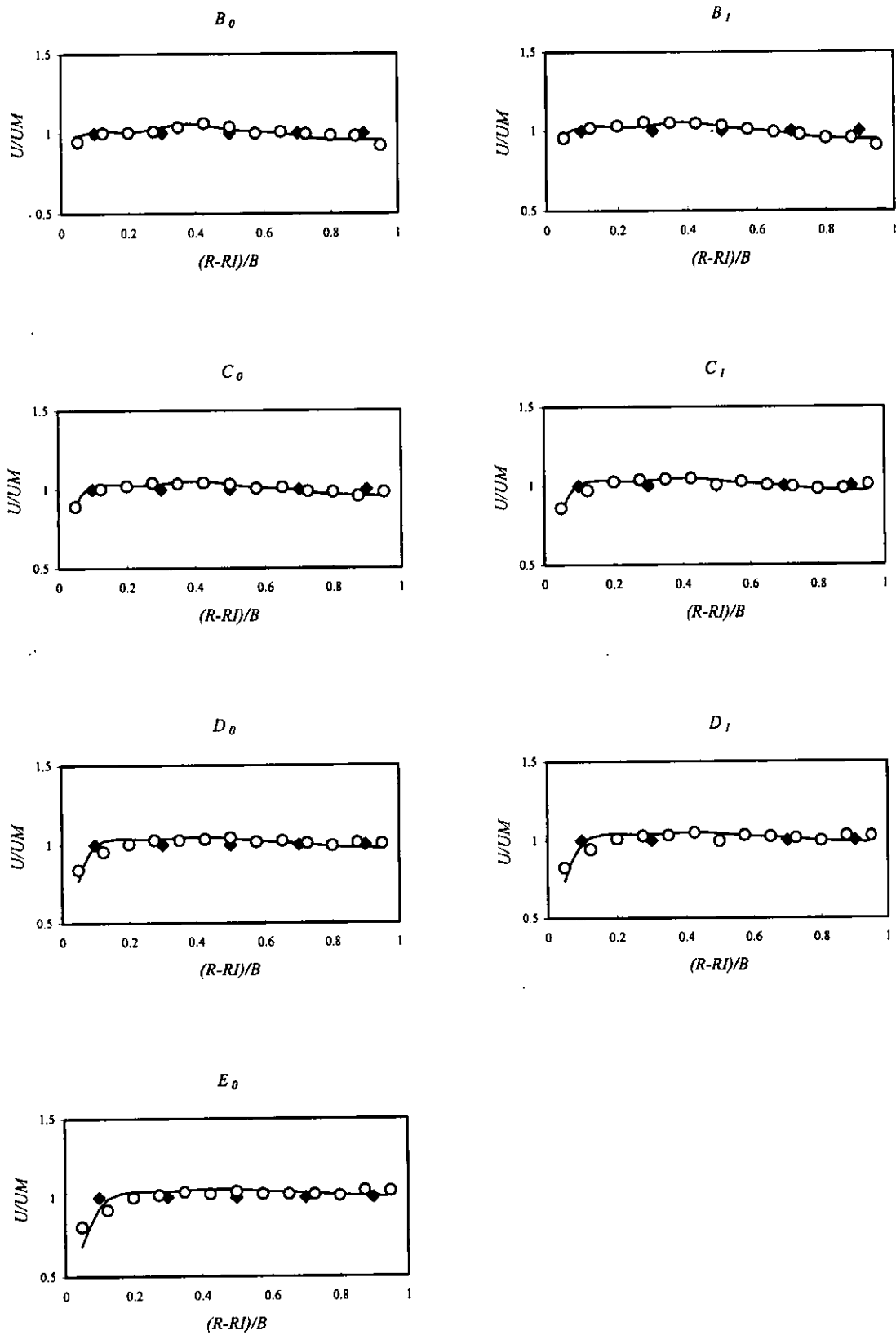


圖 6-6 U/UM 比較圖。實驗值(○)；水庫沉滓運移模式(—)；GSTARS 3.0(◆)

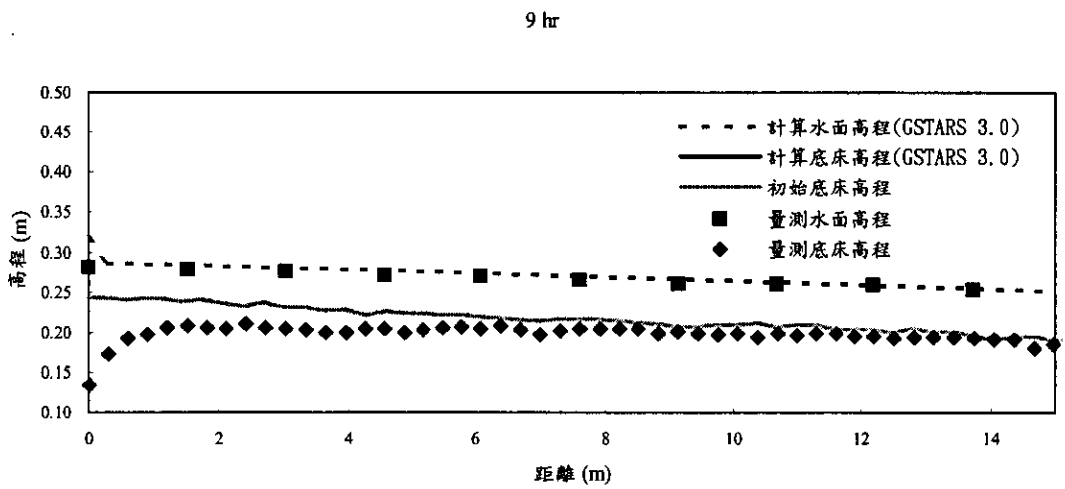
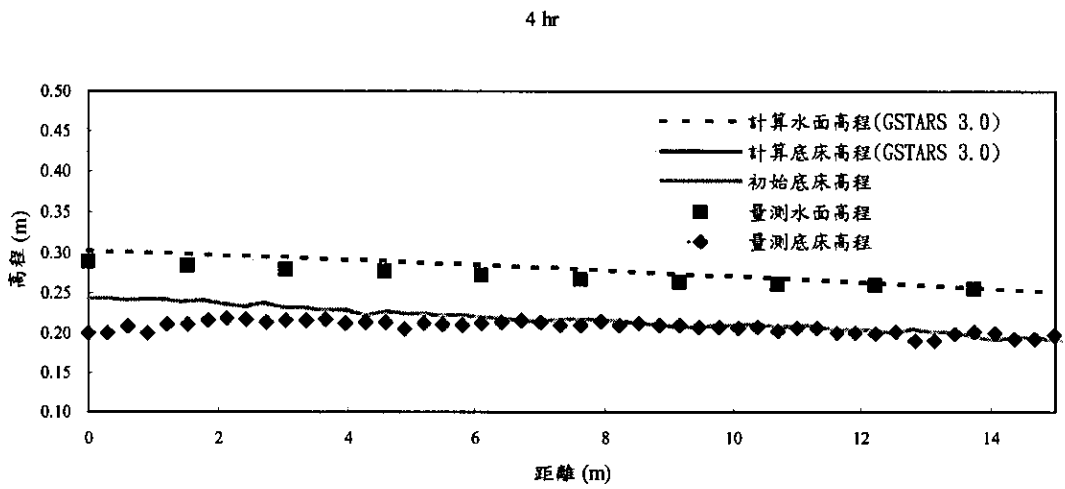
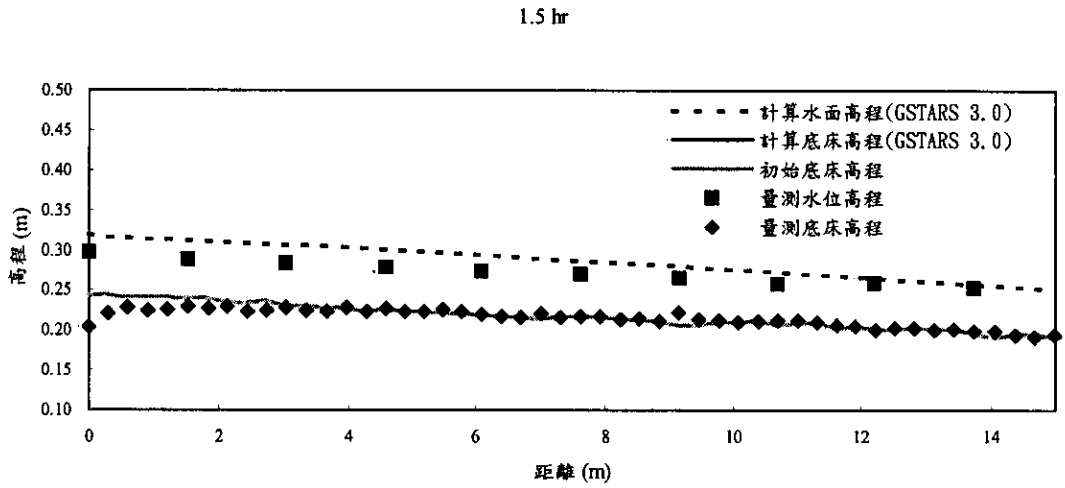
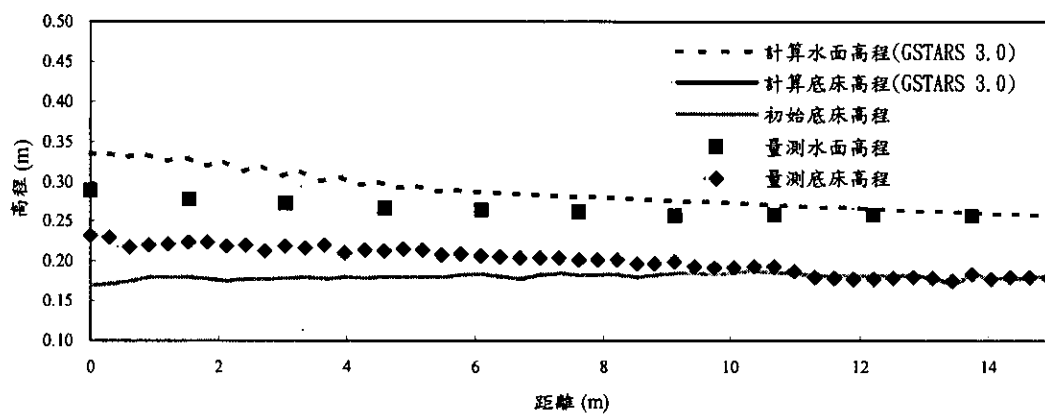


圖 6-8 沖刷案例之底床沖刷時變圖

7 hr



10 hr

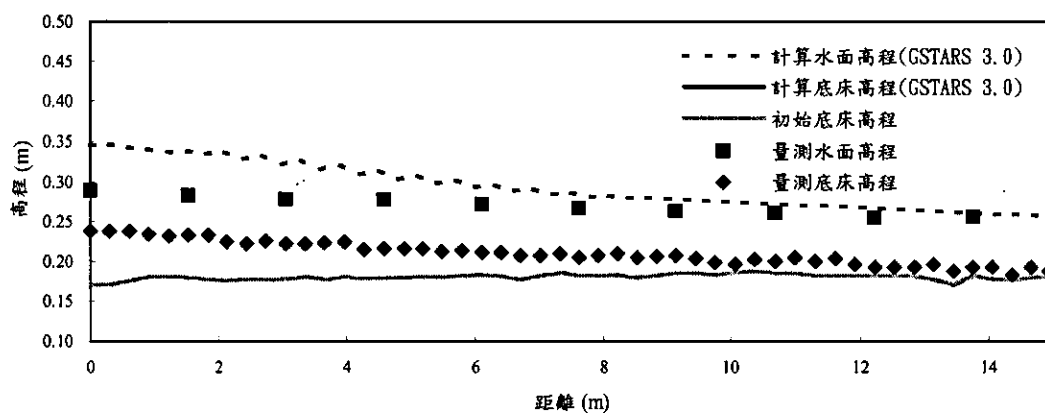


圖 6-9 淤積案例之底床沖刷時變圖(2)

第七章 NETSTARS模式英文使用手冊

NETSTARS 模式英文使用手冊之內容可分為五個部分，第一部份為模式簡介(Introduction)，敘述模式發展的目的與功能；第二部分為水理演算(Flow Routing)，介紹模式水理演算的理論基礎；第三部分為輸砂演算(Sediment Routing)，介紹模式輸砂演算的理論基礎；第四部份為資料需求(Data Requirements)，說明模式的輸入與輸出資料；第五部分為輸入資料卡(Input Data Records)，說明輸入資料卡之內容與格式。NETSTARS 模式英文使用手冊之內容請參閱附錄四所示。

第八章 結論與未來工作

8.1 結論

1. 完成二維水庫沉滓運移模式的建構。
2. 利用迴水演算、變量流、彎道、及環流流場等案例，進行二維水庫沉滓運移模式中水理子模式的測試工作，其模擬結果驗證水理子模式的正確性與應用至石門水庫之適用性。
3. 利用沖刷、淤積及沖淤交替等案例，進行二維水庫沉滓運移模式中非凝聚性沉滓子模式的測試工作，其模擬結果驗證非凝聚性沉滓子模式的正確性與實用性。
4. 根據 GSTARS 3.0 模式功能的評估結果，分別選用混合流況、凝聚性沉滓輸砂、流管間沉滓相互傳輸、最小功率法應用、支流入流及水庫演算等案例，進行 GSTARS 3.0 模式的測試工作，其模擬結果均展現合理之結果，驗證該模式實際應用的可行性。
5. 檢視變量流、彎道及環流流場案例之模擬結果，GSTARS 3.0 模式應用之石門水庫應有其限制存在。
6. 完成 NETSTARS 模式英文使用手冊。
7. 初步完成石門水庫相關資料的蒐集，根據蒐集資料之整理分析，石門水庫做為水庫沉滓運移模式之應用案例應是可行的。

8.2 未來工作

下年度計畫未來工作項目包括

1. 二維水庫沉滓運移模式凝聚性沉滓子模式之測試與驗證。
2. GSTARS 3.0 模式的評估測試工作。

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附錄一

水庫沉滓運移模式之理論基礎

附錄一 水庫沉滓運移模式之理論基礎

A.1 曲線座標

首先定義卡式座標與正交曲線(orthogonal curvilinear coordinate)座標系統的符號設定為

$$\text{卡式座標} \quad x^i = (x^1, x^2, x^3) = (x, y, z)$$

$$\text{正交曲線座標} \quad \xi^i = (\xi^1, \xi^2, \xi^3) = (\xi, \eta, z)$$

水理及輸砂控制方程式可以張量的形式表示為

(1) 水理連續方程式

$$V_j^j = 0 \quad (\text{A-1})$$

(2) 水理動量方程式

$$\frac{\partial V^i}{\partial t} + (V^i V^j)_{,j} = \frac{1}{\rho} F^i - \frac{1}{\rho} g^i P_{,j} + \frac{1}{\rho} \tau_{,j}^i \quad (\text{A-2})$$

(3) 懸浮載之質量守恆方程式

$$\frac{\partial C}{\partial t} + (C V^j)_{,j} = -\frac{1}{\rho} q_{,j}^j \quad (\text{A-3})$$

(4) 作用層之質量守恆方程式

$$\rho_s(1-p) \frac{\partial (\beta E_m)}{\partial t} + (q_b)_{,j}^j + S - S_f = 0 \quad (\text{A-4})$$

(5) 整體河床輸砂之質量守恆方程式

$$\rho_s(1-p) \frac{\partial z_b}{\partial t} + \sum [(q_b)_{,j}^j + S] = 0 \quad (\text{A-5})$$

以上諸式中， V^i = 卡式座標系統下之速度； ρ = 流體密度； F = 微體力 (body force)； P = 壓力； τ = 層流及亂流剪應力 (shear stress)。

另外，

$$\frac{\partial}{\partial \xi^i} [h_i h_j V(i)] + \frac{\partial}{\partial \xi^j} [h_i h_k V(j)] + \frac{\partial}{\partial \xi^k} [h_i h_j V(k)] = 0 \quad (\text{A-17})$$

(2) 水理動量方程式

$$\begin{aligned} & \frac{\partial V(i)}{\partial t} + \frac{V(i)}{h_i} \frac{\partial V(i)}{\partial \xi^i} + \frac{V(j)}{h_j} \frac{\partial V(i)}{\partial \xi^j} + \frac{V(k)}{h_k} \frac{\partial V(i)}{\partial \xi^k} \\ & + \left[\frac{1}{h_i h_j} \frac{\partial h_i}{\partial \xi^i} V(i) - \frac{1}{h_j h_i} \frac{\partial h_j}{\partial \xi^i} V(j) \right] V(j) \\ & + \left[\frac{1}{h_i h_k} \frac{\partial h_i}{\partial \xi^k} V(i) - \frac{1}{h_k h_i} \frac{\partial h_k}{\partial \xi^i} V(k) \right] V(k) \\ & = \frac{h_i}{\rho} F(i) - \frac{1}{\rho h_i} \frac{\partial P}{\partial \xi^i} + \frac{1}{\rho h_i} \frac{\partial \tau(ii)}{\partial \xi^i} + \frac{1}{\rho h_j} \frac{\partial \tau(ij)}{\partial \xi^j} + \frac{1}{\rho h_k} \frac{\partial \tau(ik)}{\partial \xi^k} \\ & + \left(\frac{1}{h_j h_i} \frac{\partial h_j}{\partial \xi^i} + \frac{1}{h_k h_i} \frac{\partial h_k}{\partial \xi^i} \right) \frac{\tau(ii)}{\rho} - \frac{1}{h_j h_i} \frac{\partial h_j}{\partial \xi^i} \frac{\tau(jj)}{\rho} - \frac{1}{h_k h_i} \frac{\partial h_k}{\partial \xi^i} \frac{\tau(kk)}{\rho} \\ & + \left(\frac{2}{h_j h_j} \frac{\partial h_i}{\partial \xi^j} + \frac{1}{h_k h_j} \frac{\partial h_k}{\partial \xi^j} \right) \frac{\tau(ij)}{\rho} + \left(\frac{2}{h_i h_k} \frac{\partial h_i}{\partial \xi^k} + \frac{1}{h_j h_k} \frac{\partial h_j}{\partial \xi^k} \right) \frac{\tau(ik)}{\rho} \end{aligned} \quad (\text{A-18})$$

(3) 懸浮載之質量守恆方程式

$$\begin{aligned} & h_i h_j h_k \frac{\partial C}{\partial t} + \frac{\partial}{\partial \xi^i} [h_j h_k C V(i)] + \frac{\partial}{\partial \xi^j} [h_i h_k C V(j)] + \frac{\partial}{\partial \xi^k} [h_i h_j C V(k)] \\ & = - \frac{\partial}{\partial \xi^i} \left[\frac{h_j h_k}{\rho} q(i) \right] - \frac{\partial}{\partial \xi^j} \left[\frac{h_i h_k}{\rho} q(j) \right] - \frac{\partial}{\partial \xi^k} \left[\frac{h_i h_j}{\rho} q(k) \right] \end{aligned} \quad (\text{A-19})$$

(4) 作用層之質量守恆方程式

$$\rho_s (1-p) h_i h_j \frac{\partial (\beta E_m)}{\partial t} + \frac{\partial}{\partial \xi^i} [h_j q_b(i)] + \frac{\partial}{\partial \xi^j} [h_i q_b(j)] + S - S_f = 0 \quad (\text{A-20})$$

(5) 整體河床輸砂之質量守恆方程式

$$\rho_s (1-p) h_i h_j \frac{\partial z_b}{\partial t} + \sum \left\{ \frac{\partial}{\partial \xi^i} [h_j q_b(i)] + \frac{\partial}{\partial \xi^j} [h_i q_b(j)] + S \right\} = 0 \quad (\text{A-21})$$

$$-\frac{1}{\rho h_1 h_2 d} \frac{\partial h_1}{\partial \eta} T_{11} + \frac{1}{\rho h_1 h_2 d} \frac{\partial h_2}{\partial \xi} T_{12} - \frac{\tau_b}{\rho d} + \frac{1}{\rho h_1 h_2 d} \left[-(h_2 \tau_{12})_s \frac{\partial z_s}{\partial \xi} + (h_2 \tau_{12})_b \frac{\partial z_b}{\partial \xi} - (h_1 \tau_{22})_s \frac{\partial z_s}{\partial \eta} + (h_1 \tau_{22})_b \frac{\partial z_b}{\partial \eta} \right] \quad (\text{A-24})$$

式中，

$$T_{11} = \int_b^s [\tau_{11} - \rho \overline{u'^2} - \rho(\bar{u} - \bar{u})^2] dz \quad (\text{A-25})$$

$$T_{22} = \int_b^s [\tau_{22} - \rho \overline{v'^2} - \rho(\bar{v} - \bar{v})^2] dz \quad (\text{A-26})$$

$$T_{12} = T_{21} = \int_b^s [\tau_{12} - \rho \overline{u'v'} - \rho(\bar{u} - \bar{u})(\bar{v} - \bar{v})] dz \quad (\text{A-27})$$

以上諸式中， ξ 、 η = 平面上兩正交曲線座標方向； h_1 = ξ 方向轉換係數； h_2 = η 方向轉換係數； u = ξ 方向速度； v = η 方向速度； d = 水深； g = 重力加速度； t = 時間； z_b = 底床高程； z_s = 水面高程； τ_s 、 τ_b = 底床剪應力在 ξ 與 η 方向之分量； $(\bar{\quad})$ = 時間平均； $(\bar{\quad})$ = 水深平均； (\cdot) = 時間平均瞬時擾動量；下標 s 、 b 分別代表變數在水面與底床的值； T_{11} 、 T_{12} 、 T_{22} = 有效剪應力項，包含層流剪應力 $(\tau_{11}$ 、 τ_{22} 、 $\tau_{12})$ 、亂流剪應力 $(-\rho \overline{u'^2}$ 、 $-\rho \overline{v'^2}$ 、 $-\rho \overline{u'v'})$ 與延散剪應力(dispersion stresses)。

A.2.1.2 輔助關係式

(1) 底床剪應力

底床剪應力採用 Rastogi and Rodi (1978) 之經驗式

$$\tau_b = C_f \rho \bar{u} (\bar{u}^2 + \bar{v}^2)^{1/2} \quad (\text{A-28})$$

$$\tau_b = C_f \rho \bar{v} (\bar{u}^2 + \bar{v}^2)^{1/2} \quad (\text{A-29})$$

式中， $C_f = g/c^2$ = 摩擦係數； c = Chezy 係數。

(2) 層流與亂流剪應力

以上諸式中， $r =$ 曲率半徑； $\zeta = (z - z_b)/d =$ 距離底床之高度與水深之比值。

A.2.2 輸砂部份

A.2.2.1 控制方程式

輸砂子模式將輸砂通量分離為非均勻質之懸浮載與河床載，包含某一粒徑之懸浮載質量守恆方程式、某一粒徑於作用層(active layer)之質量守恆方程式及整體河床輸砂之質量守恆方程式，分別表示如下：

(1) 對某一粒徑懸浮載之質量守恆方程式

$$\begin{aligned} \frac{\partial \bar{C}}{\partial t} + \frac{\bar{u}}{h_1} \frac{\partial \bar{C}}{\partial \xi} + \frac{\bar{v}}{h_2} \frac{\partial \bar{C}}{\partial \eta} = & -\frac{1}{\rho h_1 h_2 d} \frac{\partial}{\partial \xi} (h_2 Q_{1d}) \\ & -\frac{1}{\rho h_1 h_2 d} \frac{\partial}{\partial \eta} (h_1 Q_{2d}) + \frac{S}{\rho d} \end{aligned} \quad (\text{A.37})$$

(2) 對某一粒徑於作用層之質量守恆方程式

$$\rho_s (1-p) h_1 h_2 \frac{\partial (\beta E_m)}{\partial t} + \frac{\partial}{\partial \xi} (h_2 q_{b_1}) + \frac{\partial}{\partial \eta} (h_1 q_{b_2}) + S - S_f = 0 \quad (\text{A.38})$$

(3) 整體河床輸砂之質量守恆方程式

$$\rho_s (1-p) h_1 h_2 \frac{\partial z_b}{\partial t} + \sum \left[\frac{\partial}{\partial \xi} (h_2 q_{b_1}) + \frac{\partial}{\partial \eta} (h_1 q_{b_2}) + S \right] = 0 \quad (\text{A.39})$$

以上諸式中， $C =$ 懸浮質濃度； $\rho_s =$ 泥砂密度； $\beta =$ 粒徑百分比； $p =$ 孔隙率； $Q_i = i$ 方向某一粒徑懸浮載之擴散(diffusion)及延散(dispersion)通量； $E_m =$ 作用層厚度(active-layer thickness)； $q_{b_i} = i$ 方向某一粒徑之河床載通量(bedload flux)； $S =$ 懸浮載源(suspended-load source)； $S_f =$ 作用層源(active-layer floor source)。

A.2.2.2 輔助關係式

懸浮載源是由懸浮質向下之通量與底床亂流剪力作用產生河床質向上之通量交互作用之結果。使懸浮質下移到河床表面，主要是受到重力的影響。對某一粒徑 k 之懸浮質而言，其向下之通量可表為：

$$q_{d_k} = -\rho w_{fk} C_{d_k} \quad (\text{A.42})$$

式中， $C_{d_k} = [3.25 + 0.55 \ln(\frac{w_{fk}}{\kappa u_*})] C_k$ (Lin 1984)； C_k ：顆粒 k 之深度平均濃度； w_{fk} = 顆粒 k 之沉降速度。

另一方面，床面沈澱成為懸浮質，主要受到底床之亂流作用所造成。對某一粒徑 k 而言，河床質向上之通量可表為：

$$q_{e_k} = \rho w_{fk} \beta_k C_{e_k} \quad (\text{A.43})$$

式中， $C_{e_k} = 0.015 \frac{D_k}{a} \frac{T_k^{1.5}}{D_k^{0.3}}$ (Van Rijn 1984)； a ：沙丘高度之一半。

故由(A.42)及(A.43)式知，對某一粒徑 k 之懸浮載源可表為：

$$S_k = \rho(w_{fk} \beta_k C_{e_k} - w_{fk} C_{d_k}) \quad (\text{A.44})$$

(ii) 凝聚性沉澱

對某一顆粒 k ，向下沉澱之通量(S_d)採用Teisson (1991)之經驗式

$$S_d = P w_{fk} C_k \quad (\text{A.45})$$

式中， $P = 1 - \tau_b / \tau_{crd}$ = 沉降機率； τ_{crd} = 臨界沉降剪應力(critical shear stresses for deposition)。

對某一顆粒 k 河床質向上之通量 S_e ，採用Partheniades (1965)之沖刷經驗式

$$S_e = \beta_k M' \left(\frac{\tau_b}{\tau_{cre}} - 1 \right) \quad \text{for } \tau_b > \tau_{cre} \quad (\text{A.46})$$

(gradient theorem)化簡為：

$$Q_1 = \varepsilon_1 \frac{\partial \bar{C}}{\partial \xi} \quad (\text{A.51})$$

$$Q_2 = \varepsilon_2 \frac{\partial \bar{C}}{\partial \eta} \quad (\text{A.52})$$

式中，

$$\varepsilon_1 = 5.93u.d \quad (\text{Elder 1959}) \quad (\text{A.53})$$

$$\varepsilon_2 = \begin{cases} 0.23u.d & \text{for 直渠道 (Elder 1959)} \\ 25\left(\frac{\bar{u}d}{u.r}\right)^2 u.d & \text{for 彎道 (Fischer et al. 1979)} \end{cases} \quad (\text{A.54})$$

式中， $\varepsilon_i = i$ 方向之擴散及延散係數。

A.3 數值方法

A.3.1 水理部份

A.3.1.1 雙階操作分割趨近法

本研究基於分割操作(split-operator approach)之觀念，將動量方程式分割成二個步驟求解，分別稱為延散步驟及傳播步驟。延散步驟求解移流項和擴散項，擴散步驟求解壓力項、底床剪應力量項和連續方程式。據此，水理控制方程可改寫成

延散步驟

$$\left(\frac{\partial V}{\partial t}\right) = -(V \cdot \nabla)V + \frac{1}{\rho} \nabla \cdot T \quad (\text{A-55})$$

擴散步驟

$$\left(\frac{\partial V}{\partial t}\right) - \left(\frac{\partial V}{\partial t}\right) = -g\nabla(z_b + d) - \frac{\tau_b}{\rho d} \quad (\text{A-56})$$

$$h_1 h_2 \frac{\partial d}{\partial t} + \frac{\partial}{\partial \xi} \left(\alpha_1 \frac{\partial(\Delta d)}{\partial \xi} + \beta_1 \Delta d + \gamma_1 \right) + \frac{\partial}{\partial \eta} \left(\alpha_2 \frac{\partial(\Delta d)}{\partial \eta} + \beta_2 \Delta d + \gamma_2 \right) = 0 \quad (\text{A-63})$$

其中

$$\alpha_1 = -\frac{h_2 g \Delta t}{C_r h_1} d^n; \quad \beta_1 = \frac{h_2}{C_r} \bar{u}^{n+\frac{1}{2}} - \frac{h_2 g \Delta t}{C_r h_1} \left[\frac{\partial z_b^{n+1}}{\partial \xi} + \frac{\partial d^n}{\partial \xi} \right]; \quad \gamma_1 = \beta_1 d^n \quad (\text{A-64})$$

$$\alpha_2 = -\frac{h_1 g \Delta t}{C_r h_2} d^n; \quad \beta_2 = \frac{h_1}{C_r} \bar{v}^{n+\frac{1}{2}} - \frac{h_1 g \Delta t}{C_r h_2} \left[\frac{\partial z_b^{n+1}}{\partial \eta} + \frac{\partial d^n}{\partial \eta} \right]; \quad \gamma_2 = \beta_2 d^n \quad (\text{A-65})$$

$$C_r = 1 + \Delta t \frac{C_f \sqrt{(\bar{u}^{n+\frac{1}{2}})^2 + (\bar{v}^{n+\frac{1}{2}})^2}}{d^n} \quad (\text{A-66})$$

式中， $\Delta d = d^{n+1} - d^n$ ； $\Delta t = t^{n+1} - t^n$ 。

A.3.1.2 數值差分

在數值差分方法選用的考量上，利用顯式數值方法求解時，演算時間間隔將會受到很大的限制，在模擬天然明渠水流問題時將耗費冗長的演算時間與龐大的電腦計算量，在應用上有其困難存在。為解決這個問題，本計畫採用隱式數值方法求解。

本模式採用控制體積(control volume)法的觀念來離散控制方程式，控制體積法的基本概念如圖A-1所示，其中(a)圖為實際區域，(b)圖為計算區域，E、W、N、S 表相鄰格點，e、w、n、s 表控制面。在水理控制方程式中，除了移流項採用一階精度上風法(upwind scheme)差分外，所有空間差分均採用二階精度的中央差分法。另外，時間項則採用簡單的前項差分。

中央差分法可表示成

$$\left(\frac{\partial \Psi}{\partial \xi} \right)_p^{n+1} = \frac{\Psi_e^{n+1} - \Psi_w^{n+1}}{\Delta \xi} \quad (\text{A-67})$$

本模式目前考量三種邊界條件設定，分別為渠道入流、渠道出流與固體邊界。一般而言，渠道入流邊界條件設定為單位寬度入流量，渠道出流邊界條件則採用水位高程設定。在固體邊界處，應用側壁理論(law of the wall)，設定靠近固體邊界的邊界條件為

$$\frac{u_w}{U_*} = \frac{1}{k} \ln(Ey^+) \quad (\text{A-77})$$

其中， u_w =近固體邊界的水深平均速度； E =糙度因子=9.0； $y^+ = y_w U_* / \nu$ ， y_w =固體邊界與鄰近固體邊界格點的距離。

A.3.2 輸砂部份

A.3.2.1 結合演算法

假定底床是由 k 種粒徑所組成，則與輸砂有關之控制方程式包含 k 個深度平均懸浮載之質量守恆方程式【(A.37)式】、 k 個作用層質量之守恆方程式【(A.38)式】及一個整體河床輸砂之質量守恆方程式【(A.39)式】。求解的未知變數為 k 個深度平均懸浮載濃度 C 、 k 個粒徑百分比 β 及河床高程 z_b 。

由於各物理量具有高度之相關性，如河床質與懸浮質藉著懸浮載源 S 連結，河床高程 z_b 及作用層內之粒徑百分組成 β 又因懸浮載源 S 而變動等，故有賴結合演算方式以求解此三條方程式。(A.38)及(A.39)式為雙曲線偏微分方程式，但(A.37)式為雙曲線-橢圓形混合之偏微分方程式，因此聯立解此三方程式時，須先對(A.37)式作一適當處理。(A.37)式為延散方程式，解此方程如同解水流運動方程式，可分成下列二個步驟：

(1)移流及反應(advection-reaction)步驟

$$\frac{\partial \bar{C}}{\partial t} + \frac{\bar{u}}{h_1} \frac{\partial \bar{C}}{\partial \xi} + \frac{\bar{v}}{h_2} \frac{\partial \bar{C}}{\partial \eta} = \frac{D\bar{C}}{Dt} \quad (\text{A.82})$$

且(A.57)式變為：

$$\frac{D\bar{C}}{Dt} = \frac{S}{\rho d} \quad (\text{A.83})$$

在正交曲線座標上，其方向軌跡為：

$$\frac{h_1 d\xi}{dt} = \bar{u} \quad \text{and} \quad \frac{h_2 d\eta}{dt} = \bar{v} \quad (\text{A.84})$$

圖 A-2 為質點運動軌跡之示意圖，A 點為 $t^{n+1} = (n+1)\Delta t$ 時之到達點，D 點為 t^n 時之離開點。要解(A.83)式之全微分項，必須先解(A.84)式，即求出其積分路徑。由於模式採用固定格點，故離開點不能保證剛好落在格點上，而必須以鄰近格點採內差法求之。

由於河床載方程式與懸浮載方程式在時間尺度上相差很大，即河床質移動量遠小於懸浮質移動量。在必須聯立求解前提下，移流及反應步驟之懸浮載方程式【(A.78)式】必須使用較大之可蘭數(Courant number)，但大可蘭數將導至軌跡線必須穿過若干個格網方能落在 t^n 平面上，為求出質點之移動軌跡，路徑求解採分段方式處理，如圖 A-3 所示。

(1) 運算軌跡之計算

對某一粒徑，如以從 D_i 積至 A_i 之軌跡為例，則：

$$\xi_D^{(m+1)} = \xi_A - \frac{(u_A + u_D^{(m)})}{2h_{1M}^{(m)}} (t_A - t_D^{(m+1)}) \quad (\text{A.85})$$

$$\eta_D^{(m+1)} = \eta_A - \frac{(v_A + v_D^{(m)})}{2h_{2M}^{(m)}} (t_A - t_D^{(m+1)}) \quad (\text{A.86})$$

$$t_D^{(m+1)} = t_A - \min \left[\frac{h_{1M}^{(m)} (\xi_D^{(m)} - \xi_A)}{\frac{1}{2}(u_A + u_D^{(m)})}, \frac{h_{2M}^{(m)} (\eta_D^{(m)} - \eta_A)}{\frac{1}{2}(v_A + v_D^{(m)})} \right] \quad (\text{A.87})$$

式中，下標 A,D 表示在到達和離去點的軌跡，M 為 A 與 D 之中點值；上標 m 表疊代次數。此疊代過程係為求得離去點 D_i 之座標值。

$$\left[\frac{\partial F_{2k+1}}{\partial \bar{s}}\right]\Delta\bar{s} = -F_{2k+1}({}^m\bar{s}^{n+1}) \quad k=1, K \quad (\text{A.96})$$

式中， $\left[\frac{\partial F}{\partial \bar{s}}\right]$ 為 Jacobian 係數矩陣中之一列向量； ${}^m\bar{s}^{n+1}$ 為前一次疊代未知向量； $\Delta\bar{s}$ 為疊代修正向量，可表為 $\Delta\bar{s} = (\Delta s_1, \Delta s_{2k}, \Delta s_{2k+1})$ 。解得修正向量 $\Delta\bar{s}$ 後，可得新的 ${}^{m+1}\bar{s}^{n+1}$ 向量：

$${}^{m+1}\bar{s}^{n+1} = {}^m\bar{s}^{n+1} + \Delta\bar{s} \quad (\text{A.97})$$

當達到下列之收斂準則，疊代得以結束。

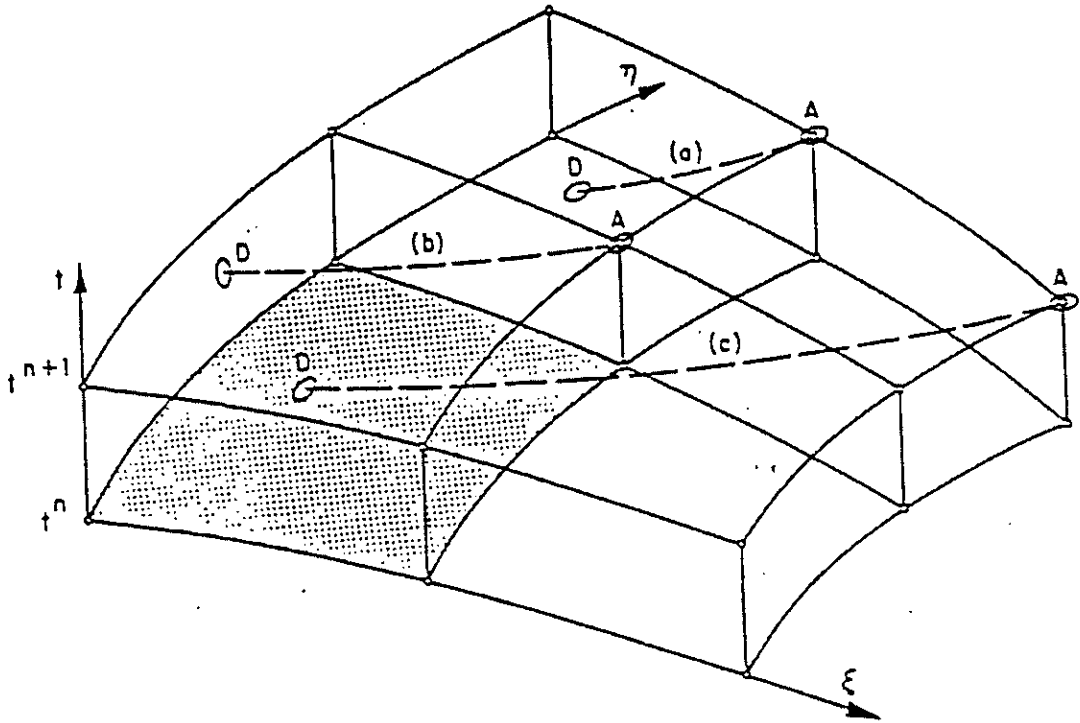


圖 A-2 質點運移軌跡示意圖

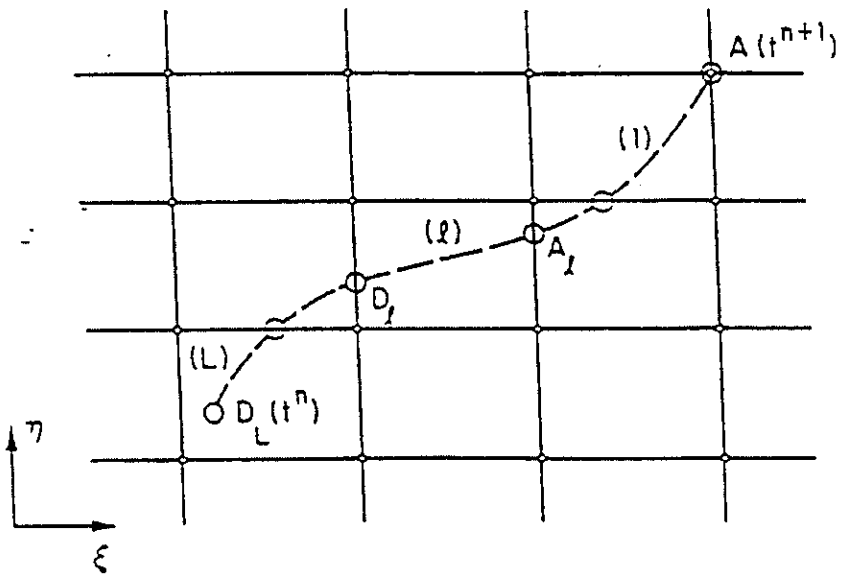


圖 A-3 質點運移軌跡在計算平面之示意圖

附錄二

GSTARS 3.0 模式理論基礎

附錄二 GSTARS 3.0 模式理論基礎

B.1 流線與流管

流線與流管概念的應用為 GSTARS 3.0 模式發展之特色之一。流線(streamline)係指流場中的一組曲線在每一瞬間(t)，其任一點的切線向量平行於該點的速度向量，該組曲線係稱為流線。而流管(stream tube)係指以流線為邊壁所組成的流管概念。因流體質點不可能穿透流管邊界(即流線)，所以沿著每一流管，其流量是不變的；且其流速與流管面積成反比。圖 B-1 所示為 GSTARS 3.0 模式所應用之流管基本概念。

對一定量不可壓縮流體而言，沿著一流管之總能量 H_t 為一常數：

$$\frac{P}{\gamma} + \frac{v^2}{2g} + h = H_t = \text{constant} \quad (\text{B-1})$$

式中， P =作用於橫斷面之壓力； γ =水之單位重； v =流速； g =重力加速度；及 h =水力高度。

B.2 迴水演算

對一擬似定量流而言，流量歷線可以由一連串定常流量變化歷線組成來表示，如圖 B-2 所示。在每一段定常流量裡，即以定量流方程式進行迴水演算。GSTARS 3.0 模式係以標準步推法(standard-step method)求解能量方程式。為解決不同流況間之轉換，如超臨界流轉換至亞臨界流流況，即水躍發生之情況，則以動量方程式來替代能量方程式。

B.2.1 能量方程式

GSTARS 3.0 模式所使用之能量方程式：

所以水力篩選(sorting)及護甲(armoring)作用的演算工作亦在每一流管中獨立進行。故由每一演算時刻的不同流管中輸砂的演算結果，便可獲得不同時刻每一斷面上輸砂特性的側向變化。而 GSTARS 3.0 模式因其並非真正二維模式，故對於渦流(eddies)、環流(recirculating flow)、二次流(secondary flow)及側向水面線變化等現象無法模擬。

B.4 輸砂演算與渠道幾何形狀調整

B.4.1 泥砂連續方程式

對一維變量流中輸砂演算中最基礎的泥砂連續方程式可表為：

$$\frac{\partial Q_s}{\partial x} + \eta \frac{\partial A_d}{\partial t} + \frac{\partial A_s}{\partial t} - q_{lat} = 0 \quad (B-4)$$

式中， η = 單位河床層中泥砂體積； A_d = 單位長度中床砂體積； A_s = 橫斷面上單位長度懸移泥砂體積； Q_s = 泥砂體積流量；及 q_{lat} = 側向泥砂入流量。在 GSTARS 3.0 模式中有幾個假設，以化簡(B-4)式。首先，假設橫斷面上懸移泥砂濃度遠小於床砂的變化，亦即：

$$\frac{\partial A_s}{\partial t} \ll \eta \frac{\partial A_d}{\partial t} \quad (B-5)$$

其次，假設泥砂輸移函數中的參數在每一演算時距內，每一橫斷面上皆為常數；亦即：

$$\frac{\partial Q_s}{\partial t} = 0 \quad \text{或} \quad \frac{\partial Q_s}{\partial x} = \frac{dQ_s}{dx} \quad (B-6)$$

前述假設僅在每一橫斷面幾何形狀改變極微情況下方能成立，亦即在每一演算時距中泥砂沖淤量變化不可太大。這一假設對水、砂分離演算而言是成立的，故泥砂連續方程式最後可化簡成：

的原則劃分多個流管，在每個流管中分別計算水理參數及輸砂量，再代入輸砂連續方程式中求解每個流管內平均底床沖淤情形。GSTARS 3.0 模式中提供 14 個非凝聚性沉澱的總輸砂量公式，及一套凝聚性沉澱之輸砂公式。14 個非凝聚性沉澱輸砂公式分別為：

1. Meyer-Peter and Muller's Formula(1948)：可用於沉澱粒徑大於 10 mm 的情況，公式如下：

$$\gamma \left(\frac{K_s}{K_r} \right)^{3/2} RS = 0.047(\gamma_s - \gamma)d + 0.25\rho^{1/3}q_b^{2/3} \quad (\text{B-8})$$

式中， γ 、 γ_s 代表水和沉澱的單位重量；

R 為水力半徑；

S 為能量坡降；

d 為沉澱平均粒徑。

2. Laursen's Formula (1958) and Modification by Madden (1993)：適用於沉澱粒徑大於 0.0625 mm 時，其公式如下：

$$C_i = 0.01\gamma \sum_i p_i \left(\frac{d_i}{D} \right)^{7/6} \left(\frac{\tau'}{\tau_{ci}} - 1 \right) f \left(\frac{u_*}{\omega_i} \right) \quad (\text{B-9})$$

式中， C_i =單位體積之平均輸砂濃度重量；

p_i =泥砂粒徑分析所佔比重；

d_i =泥砂粒徑；

ω_i =粒徑 d_i 泥砂之沉降速度；

D=水深。

3. Toffaleti's Method (1969)：適用於沉澱粒徑大於 0.0625 mm 時，其公式如下：

6. Ackers and Whites' Method(1990)：針對前述第 5 項公式其對於粒徑小於 0.2 mm 的沉澱之運移速率有過度預測的缺憾，Ackers 和 White (1990)提出此修正公式。其與第 5 項公式之不同點詳如表 B-1 所示。
7. Yang's Sand (1973) and Gravel (1984) Transport Formulas：對於泥砂而言，沉澱粒徑必須小於 2 mm，其公式如下：

$$\log C_{ts} = 5.435 - 0.286 \log \frac{\omega d}{\nu} - 0.457 \log \frac{u_*}{\omega} + \left(1.799 - 0.409 \log \frac{\omega d}{\nu} - 0.314 \log \frac{u_*}{\omega} \right) \log \left(\frac{VS}{\omega} - \frac{V_{cr} S}{\omega} \right) \quad (\text{B-16})$$

式中， C_{ts} =總沉澱濃度重(ppm)；

ω =泥砂沉降速度；

ν =水之運動黏滯係數；

u_* =剪力速度；

VS =單位渠流功率；

V_{cr} =臨界啟動平均流速。

對於礫石而言，適用於平均粒徑小於 10 mm 時，其公式如下：

$$\log C_{ig} = 6.681 - 0.633 \log \frac{\omega d}{\nu} - 4.816 \log \frac{u_*}{\omega} + \left(2.784 - 0.305 \log \frac{\omega d}{\nu} - 0.282 \log \frac{u_*}{\omega} \right) \log \left(\frac{VS}{\omega} - \frac{V_{cr} S}{\omega} \right) \quad (\text{B-17})$$

式中， C_{ig} =總礫石濃度重(ppm)。

8. Yang's Sand (1979) and Gravel (1984) Transport Formulas：楊氏於 1979 年提出適用於泥砂之公式：

$$\log C_{ts} = 5.165 - 0.153 \log \frac{\omega d}{\nu} - 0.297 \log \frac{u_*}{\omega} + \left(1.780 - 0.360 \log \frac{\omega d}{\nu} - 0.480 \log \frac{u_*}{\omega} \right) \log \frac{VS}{\omega} \quad (\text{B-18})$$

12. Tsinghua University Equation for Reservoir Flushing(1985)：此公式適用於水庫排砂之情況，可表示成

$$Q_s = \Omega \frac{Q^{1.6} S_0^{1.2}}{B^{0.6}} \quad (\text{B-24})$$

式中， Q_s =輸砂量(公噸/s)； Q =流量(cms)； S_0 =底床坡度； Ω =經驗常數。

13. Ashida and Michiue Method (1972)；可適用於水庫輸砂演算，其公式可表示成

$$\frac{q_b}{\rho u_*^3 d} = 17 \tau_*^* \left(1 - \frac{\tau_c^*}{\tau_*^*}\right) \left(1 - \sqrt{\frac{\tau_c^*}{\tau_*^*}}\right) \quad (\text{B-25})$$

$$\tau_*^* = \frac{u_*^2}{Ggd} \quad (\text{B-26})$$

$$\tau_c^* = \frac{u_c^2}{Ggd} \quad (\text{B-27})$$

$$\tau_e^* = \frac{u_e^2}{Ggd} \quad (\text{B-28})$$

$$\frac{v}{u_*^*} = 5.75 \log\left(\frac{R_h / d_{50}}{1 + 2\tau_*^*}\right) + 6.0 \quad (\text{B-29})$$

式中， p =該粒徑所佔百分比； G =比重； R_h =水力半徑。

GSTARS 3.0 模式提供一套計算凝聚性沉澱(平均粒徑小於 0.0625 mm)的輸移公式。當底床剪應力 τ_b 小於底床臨界沉積剪應力 τ_{cd} 時，GSTARS 3.0 模式應用以下公式(Krone 1962)：

$$\frac{C}{C_0} = e^{-k\Delta t} \quad (\text{B-30})$$

$$k = \frac{w_s}{D} \left(1 - \frac{\tau_b}{\tau_{cd}}\right) \quad (\text{B-31})$$

在某些情況下，時間延滯性(time delay)或空間延滯性(spatial delay)效應往往是不可忽略的。例如水庫輸砂現象或河口泥砂沖淤現象。或如實驗室中經常可觀測到水庫入流清水經常要行經一段距離後才能將底泥完全沖起達飽和濃度狀態。為能模擬此種現象，GSTARS 3.0 模式應用對流—擴散方程式解析解為基礎，以計算非平衡泥砂輸移量(Han 1980)：

$$C_i = C_i^* + (C_{i-1} - C_{i-1}^*) \exp\left\{-\frac{\psi\omega_s\Delta x}{q}\right\} + (C_{i-1}^* - C_i^*) \left(\frac{q}{\psi\omega_s\Delta x}\right) \left[1 - \exp\left\{-\frac{\psi\omega_s\Delta x}{q}\right\}\right] \quad (\text{B-34})$$

式中，C=泥砂濃度；C*=泥砂輸移能力；q=單位寬度流量； Δx =計算長度； ω_s =泥砂沉降速度；i=演算斷面標示；及 Ψ =無因次參數，對於淤積情況 $\Psi=0.25$ ，對於沖刷情況 $\Psi=1.0$ (Han and He (1990))。

B.4.5 泥砂沉降速度計算

GSTARS 2.0 模式中有多種泥砂沉降速度計算方法，可依不同輸砂公式及粒徑做選擇。當採用 Toffaleti 輸砂公式時，以 Rubey 公式來計算泥砂沉降速度：

$$\omega_s = F \sqrt{dg(\gamma_s - 1)} \quad (\text{B-35})$$

其中，

$$F = \left[\frac{2}{3} + \frac{36\nu^2}{gd^3(\gamma_s - 1)} \right]^{1/2} - \left[\frac{36\nu^2}{gd^3(\gamma_s - 1)} \right]^{1/2} \quad (\text{B-36})$$

上式在泥砂粒徑介於 0.0625~1 mm 間成立；若粒徑大於 1 mm 則 $F=0.79$ 。式中， ω_s =泥砂沉速； γ_s =泥砂比重； ν =水之運動粘滯係數。至於採用 Toffaleti 公式以外之輸砂公式，則泥砂沉速可由圖 B-4 之各關係曲線來決定(U.S. Interagency Committee on Water Resources Subcommittee on Sedimentation, 1957)。

$$q_s = q_t \cos \sigma \quad (\text{B-39})$$

$$q_r = q_t \sin \sigma \quad (\text{B-40})$$

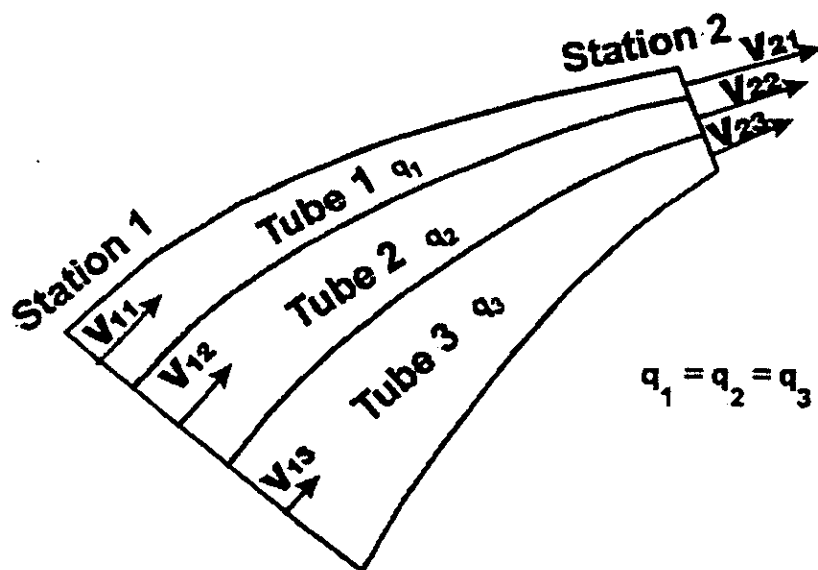


圖 B-1 流管基本概念圖

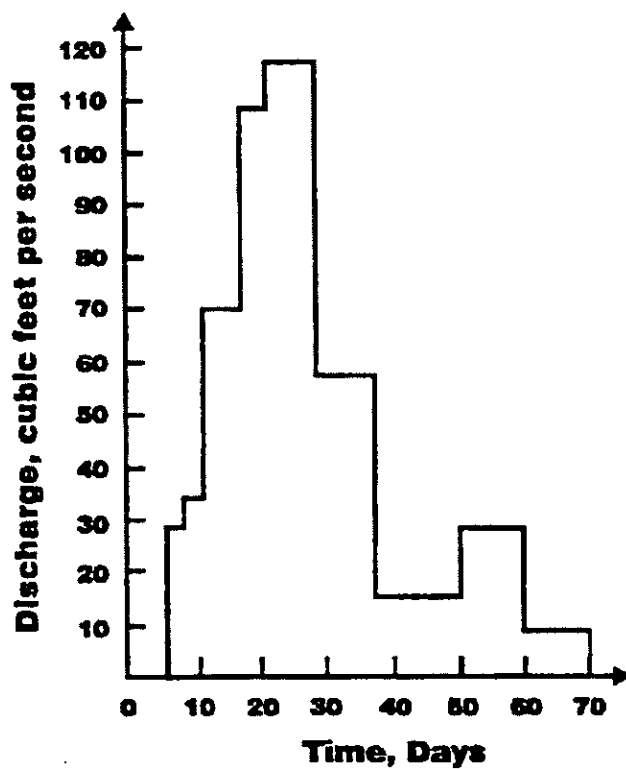


圖 B-2 流量歷線圖

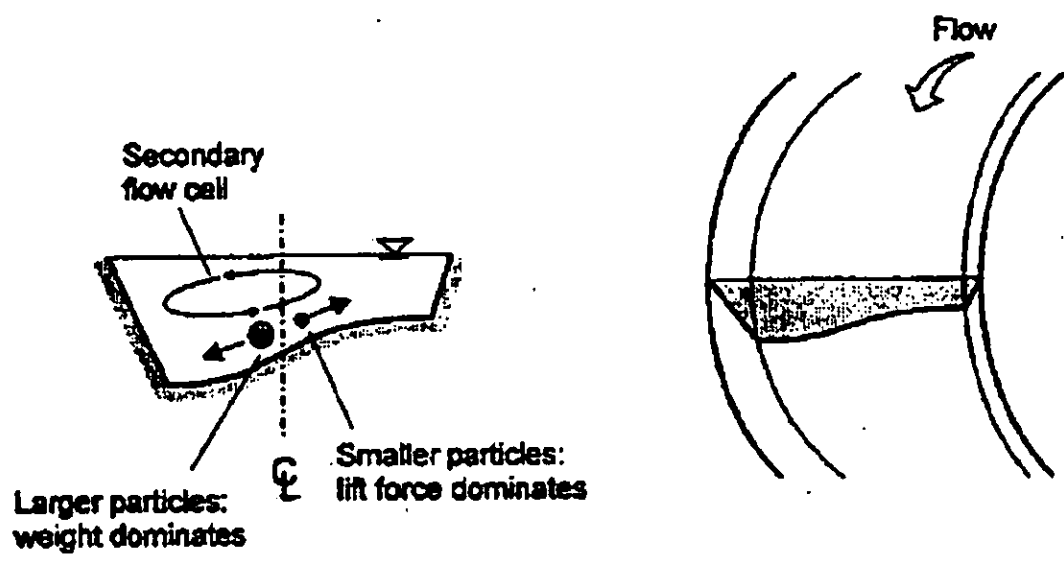


圖 B-5 沉澱側向傳輸示意圖

附錄三

GSTARS 3.0 模式輸入簡介

附錄三 GSTARS 3.0 模式輸入簡介

C.1 資料輸入格式

GSTARS 3.0 模式的資料輸入係由一系列的卡(record)組合而成，輸入資料檔中的每一行均代表一張卡的設定，如圖 C-1 所示，每一行的最大輸入長度為 80 個字元(characters)，且可區分為 11 個不同的欄位(field)。欄位 0 包含 2 個字元，用來表示卡的名稱；欄位 1 至欄位 10 用來輸入此卡所需的資料，其中欄位 1 包含 6 個字元，欄位 2 至欄位 10 各包含 8 個字元。

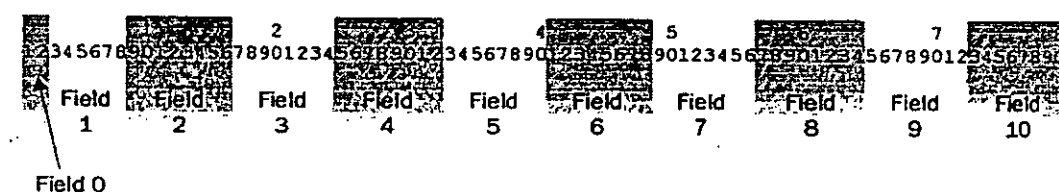


圖 C-1 資料輸入格式示意圖

C.2 水理資料

C.2.1 斷面資料

在進行數值計算時，需將演算的河段(reach)切割成若干斷面(cross section)，每一個斷面所需要建立的資料敘述如下：

1. 距最下游斷面的距離：用以描述該斷面在模擬河段的相對位置，此距離乃是沿著流線方向計算其累距。
2. 劃分斷面子區域：根據斷面幾何形狀，可將斷面區分為若干子區域。例如包含左右兩高灘地的複式斷面，即可區分為 3 個子區域，子區域 1 為左岸高灘地，子區域 2 為主深水槽，子區域 3 為右岸高灘地。
3. 斷面形狀：如圖 C-2 所示，根據斷面形狀選取適當且足夠的代表點，且

XS 卡：由左岸往右岸的方向，依序輸入代表點在側方向的位置(lateral position)與底床高程資料。GSTARS 3.0 最多允許 196 個代表點，且限制底床高程資料不可以為負值。

RH 卡：由左岸往右岸的方向，依序輸入各斷面子區域的糙度係數。RH 卡預設的糙度公式為 Manning 公式，利用 RE 卡可設定使用其他糙度公式。

C.2.2 流量與水位資料

GSTARS 3.0 在進行數值模擬時，需設定上、下游邊界條件。上游邊界條件為入流量資料，下游邊界條件為水位資料。GSTARS 3.0 可利用 QQ、SS 兩張卡設定上述流量與水位資料輸入的形式。表 C-1 描述三種流量與水位資料輸入的形式，並列出各種輸入形式設定時所需的卡。

表 C-1 流量與水位資料輸入形式一覽表

REQUIRED RECORDS		QQ
SS	TABLE OF DISCHARGES	DISCRETIZED DISCHARGES
RATING CURVE	NC, RC, TQ 形式二	NC, RC, DD 形式一
STAGE DISCHARGE TABLE	TL, SQ 形式三	

C.2.2.1 形式一

此輸入形式流量資料採用入流歷線(hydrograph)，水位資料採用率定曲線(rating curve)。舉例來說，圖 C-4 為一假定的入流歷線，首先將實際入流歷線(actual hydrograph)離散化(discretized)成階梯狀的離散化入流歷線(discretized hydrograph)，此離散化入流歷線資料為 1 個 Δt 的 Q_1 、1 個 Δt 的 Q_2 、1 個 Δt 的 Q_3 、1 個 Δt 的 Q_4 、2 個 Δt 的 Q_5 、2 個 Δt 的 Q_6 及 1 個 Δt 的 Q_7 (假設所有的 Δt 均相同，設定為 1 天)。假定下游邊界為第 23 斷面，此處的率

DISCHARGE，一為 DISCRETIZED DISCHARGES。

SS 卡：用來設定水位的輸入形式。亦有兩種形式可以選用，一為 RATING CURVE，一為 STAGE DISCHARGE TABLE。

DD 卡：用來輸入離散化入流歷線。欄位 1 為對應流量的時距數，欄位 2 為為流量值。此卡在使用上需特別注意，所有 DD 卡時距數的累加值應等於 IT 卡中的總演算時距。

NC 卡：RATING CURVE 的數目。

RC 卡：用來輸入 RATING CURVE 的公式。欄位 1 為此率定曲線應用的斷面；欄位 2 至欄位 4 為率定曲線的係數值。

C.2.2.2 形式二

形式二可視為形式一的一種特殊情況，即離散化入流歷線中每一個流量所對應的時距數均是相同的，舉例如表 C-2 所示。

表 C-2 形式二入流量資料

Days	Discharge (ft ³ /s)
0	200
1	450
2	700
3	1,020
4	1,000
5	700
6	525
7	400
8	325

假設率定曲線應用斷面及其公式與 C.2.2.1 節相同，則此案例的輸入資料可表示如圖 C-6 所示。

		2		4	
345678	901234	567890	1234	567890	1234
	9.		1.		
			DISCHARGE		
			DISCHARGE		
	35				
SQ	200.				
SQ	450.				
SQ	700.				
SQ	1020.				
SQ	1000.				
SQ	700.				
SQ	525.				
SQ	400.				
SQ	325.				

圖 C-7 形式三之輸入資料

C.3 輸砂資料

C.3.1 入砂量資料

在進行輸砂演算時，在上游邊界必須給定入砂量的相關資訊。GSTARS 3.0 提供兩種方法來設定入砂量資料，一為利用 QS 卡來輸入離散化入砂量歷線，一為利用 QR 卡來輸入輸砂率定曲線。QS 卡的輸入格式與 C.2.2.1 節的 DD 卡類似，在此不多贅述，此部份僅就 QR 卡作說明。假設已知輸砂率定曲線如下式所示

$$\text{入砂量} = 0.4Q^{1.2} \quad (\text{C-2})$$

則此時 GSTARS 3.0 的輸入資料可表示如圖 C-8 所示。

		2		4	
345678	901234	567890	1234	567890	1234
QR	0.4		1.2		

圖 C-8 QR 卡輸入格式

其中欄位 1 與欄位 2 為率定曲線的係數值。

若沒有額外設定，入砂量中各代表粒徑的百分組成分布情形則假定與上游邊界斷面底床的粒徑百分組成相同。GSTARS 3.0 亦可利用 IQ 卡與 IS

則此時 GSTARS 3.0 的輸入資料可表示如圖 C-11 所示。

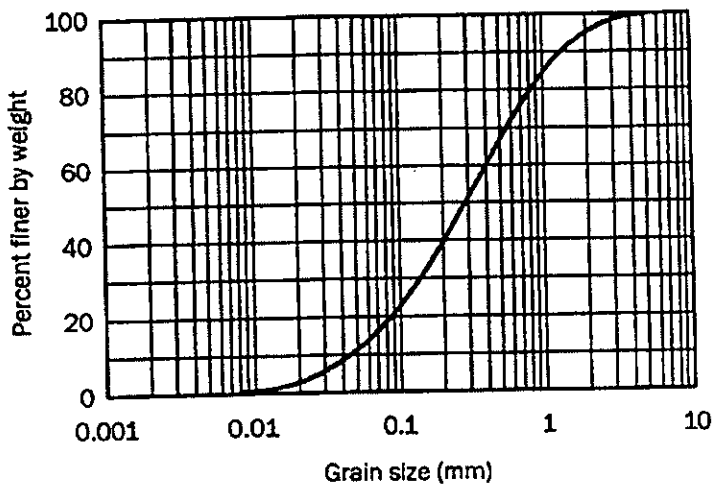


圖 C-10 模擬區域的粒徑百分組成分布圖

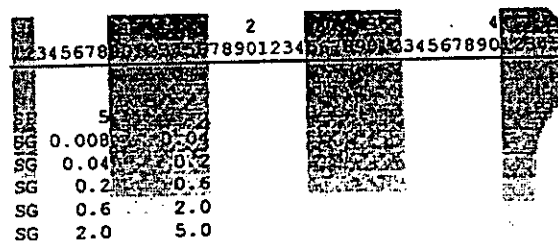


圖 C-11 模擬區域粒徑百分組成分布設定示意圖

茲將圖 C-11 中所使用到的卡說明如下：

SF 卡：欄位 1 為代表粒徑群的數目；欄位 2 為設定沉滓的乾比重(dry specific weight)，0 代表使用預設值。GSTARS 3.0 最多允許 10 個代表粒徑群。

SG 卡：欄位 1 為代表粒徑群的最小粒徑；欄位 2 為代表粒徑群的最大粒徑；欄位 3 為乾比重的設定，0 代表使用預設值。

GSTARS 3.0 利用 SD 卡來設定各斷面底床粒徑百分組成資料。舉例而言，圖 C-12 為某斷面底床粒徑百分組成分布圖(選用 5 個代表粒徑群)，則此時 GSTARS 3.0 的輸入資料可表示如圖 C-13 所示。

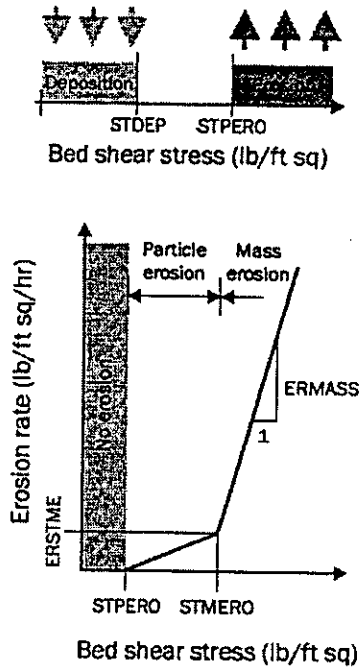


圖 C-14 凝聚性沉澱參數示意圖

STDEP：臨界沉降剪應力。

STPERO：臨界表層沖蝕剪應力。

STMERO：臨界塊狀沖蝕剪應力。

ERMAS：塊狀沖蝕沖蝕率曲線的坡度。

ERSTME：當底床剪應力等於臨界塊狀沖蝕剪應力時的沖刷率。

C.4 最小功率(stream power minimization)

GSTARS 3.0 利用 MR 卡控制最小功率概念的設定。若引用最小功率的功能，則每一個斷面均需有一張 MR 卡，輸入的順序則為上游往下游方向。舉例而言，某一河段包含 5 個斷面，寬度均為 100 ft (側向位置介於 0~100)，底床高程 1000 ft，前 3 個斷面在底床高程 920 ft 以下為不可沖刷層；第 3 個斷面在側向位置 112 ft 處有一結構物，即此處邊坡不會遭到沖蝕破壞；第 5 個斷面在側向位置 0 ft 處有一結構物，即此處邊坡不會遭到沖蝕破壞。則

```

345678          2          4
78901234          34567890
Example of general input
data file
Tributary discharge:
D1 50
D2 50
Sediment discharge:
M1 0.10 0.50
Sediment discharge:
Q1 50 0.10
Q2 50 0.10
Sediment composition:
IQ 2 0.10 100.0
IS 0.50 0.40
IS 0.40 0.40
IS 0.10 0.20

```

圖 C-17 側入流輸入檔範例

MX 卡：用以設定側入流砂量分配至各流管的比例。

C.6 水庫演算資料

GSTARS 3.0 進行水庫演算時，首先需將 QQ 卡設定為 TABLE OF DISCHARGE，SS 卡設定為 DISCHARGE AT DAM，再利用 HR 與 DR 卡設定相關資料。圖 C-18 為一假定案例水庫上游入流量與水庫出流量示意圖，其資料可離散如表 C-5 所示， $\Delta t=1$ day，總模擬時間為 9 day。圖 C-19 為此案例輸入資料。

表 C-5 水庫入流量與出流量資料

Days	Inflow Discharge	Outflow Discharge
1	200	25
2	400	200
3	625	250
4	625	325
5	300	325
6	200	325
7	200	250
8	25	200
9	25	25

PR 卡：用來設定模擬結果的輸出。欄位 1 為輸出資料的控制，-1 表不輸出資料，0 表輸出水面高程與輸砂演算紀錄，1 表除了 0 的資料外還有正常與臨界水深的輸出，2 表除了 1 的資料外還有流管的幾何與流量資料，3 表除了 2 的資料外還有輸砂量的資料。欄位 2 為資料輸出時程之控制。

PW 卡：控制斷面幾何資訊輸出的設定。

PW 卡：控制水面高程輸出的設定

附錄四

NETSTARS 模式英文使用手冊

附錄四 NETSTARS 模式英文使用手冊

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associated river bed variations by separate computations of hydraulic routing and sediment routing. Known stage hydrograph and discharge hydrograph are used to define boundary conditions and calibrate hydraulic parameters in flow computations. For a given discharge, the water surface profile may be obtained by the relationship among energy grade line, flow velocity and flow depth at each cross section. With known discharge and stage, the cross section is subdivided into several stream tubes with equal-conveyance in each tube applying the concept of stream tube. Sediment computations are performed for each stream tube to obtain sediment load and suspended sediment concentration with the aid of boundary conditions of sediment inflow and suspended concentration. Appropriate sediment transport equation may be selected for the flow conditions and sediment characteristics in question. Computational results of each time step are used to redefine the water surface elevation and bed form. The NETSTARS is, as is GSTARS, capable of modeling sorting and armoring phenomena, and may be used for beds of non-uniform sands.

1.2 MODEL APPLICATIONS

Broad applications of NETSTARS model include flow simulations of complex river systems of main stream and tributaries, with steep slope or mild slope, of hydraulic jumps, subcritical and supercritical flows in steady-state conditions, of subcritical flows in unsteady-state conditions, and corresponding scour and deposition simulations of river beds. Bed variations in the transverse direction are reflected as a result of subdividing of stream tubes in the cross section to redefine stream tube boundaries in the simulation. Backwater computation (II) in the model is able to simulate flow conditions in subcritical flows, supercritical flows and mixed flows of supercritical and supercritical types. Simulated results of longitudinal bed profile with one single stream tube are equivalent to one-dimensional HEC-6 (1993) simulations.

Sediment movements in natural streams are generally in equilibrium conditions. The equilibrium conditions no longer exist when a reservoir or hydraulic structure is erected on the river. Consequently, characteristics of flow and sediment transport in the stream change continuously as time progresses. Sediment transport formulas for total load, suitable for rivers in equilibrium state, are used to estimate total sediment load in most models. The NETSTARS separates suspended load and bed load in simulation computations and may be used to forecast changing trends in flow behavior, sediment transport and bed form of the river. Simulated results may prove helpful to designs of check dam, reservoir and diversion channel

2. FLOW ROUTING

2.1 INTRODUCTION

Computational methods for hydraulic simulations include steady-flow computation, unsteady-flow computation, backwater computation (I) and backwater computation (II). Steady-flow computation employs 1-D energy equation along with continuity equation at the node to solve for stage and discharge by finite difference techniques. Unsteady-flow computation employs de Saint Venant's 1-D momentum and continuity equations for gradually-varied flows along with continuity principle at the node using finite differences to solve for stage and discharge. Both backwater computations (I) and (II) employ 1-D energy equation on each single stream tube to solve for stages, and revise link discharges at the node until the stages from all links at the node are consistent with one another. The backwater computation (II) is capable of computing mixed flows of subcritical and supercritical types.

2.2 UNSTEADY FLOW COMPUTATION

2.2.1 Governing Equations

For unsteady flow computation, the de St Venant (1871) hypotheses are essentially taken into account in the above equations:

- (1) the flow is one-dimensional, i.e., the velocity is uniform over the cross section and the water level across the section is horizontal;
- (2) hydrostatic pressure distribution prevails at any point in the channel;
- (3) the resistance laws for steady-state flow are applicable to unsteady flow;
- (4) the channel bed slope is small

The basic one-dimensional governing equations for unsteady water flow are:

Water-Continuity Equation

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = q \quad (2.1)$$

Momentum Equation

$$\frac{\partial Q}{\partial t} + \frac{\partial}{\partial x} \left(\alpha \frac{Q^2}{A} \right) + gA \frac{\partial y}{\partial x} + gAS_f - uq = 0 \quad (2.2)$$

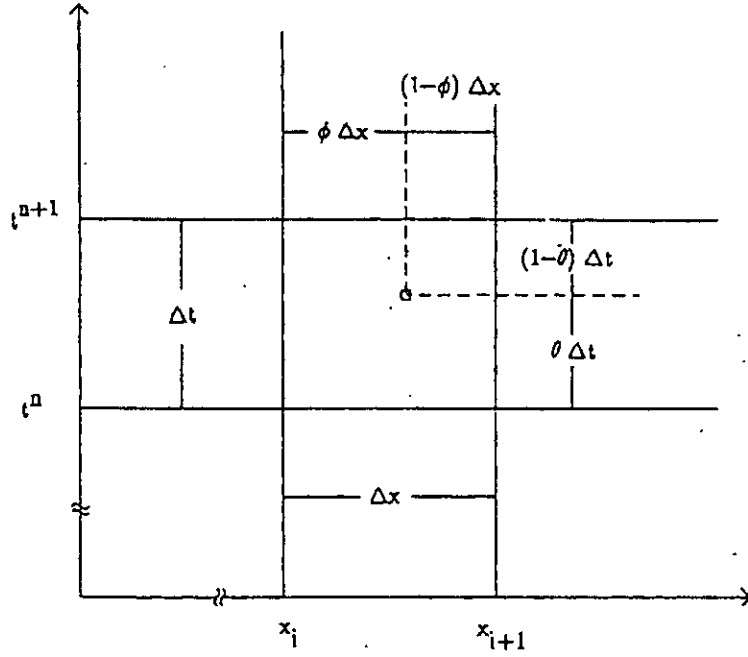


Fig. 2.1 Preissmann four point finite-difference approximation

Therefore, Eqs. (2.1) and (2.2) take the following algebraic forms, after use of the Preissmann's discretizations:

(1) Continuity equation

$$\begin{aligned} \frac{\phi}{\Delta t} (A_{i+1}^{n+1} - A_{i+1}^n) + \frac{(1-\phi)}{\Delta t} (A_i^{n+1} - A_i^n) + \frac{\theta}{\Delta x} (Q_{i+1}^{n+1} - Q_i^{n+1}) \\ + \frac{(1-\theta)}{\Delta x} (Q_{i+1}^n - Q_i^n) - q_i = 0 \end{aligned} \quad (2.9)$$

(2) Momentum equation

$$\begin{aligned} \frac{\phi}{\Delta t} (Q_{i+1}^{n+1} - Q_{i+1}^n) + \frac{(1-\phi)}{\Delta t} (Q_i^{n+1} - Q_i^n) \\ + 2 \left[\alpha \theta [(1-\phi) \frac{Q_i^{n+1}}{A_i^{n+1}} + \phi \frac{Q_{i+1}^{n+1}}{A_{i+1}^{n+1}}] + \alpha (1-\theta) [(1-\phi) \frac{Q_i^n}{A_i^n} + \phi \frac{Q_{i+1}^n}{A_{i+1}^n}] \right] \\ \left[\frac{\theta}{\Delta x} (Q_{i+1}^{n+1} - Q_i^{n+1}) + \frac{(1-\theta)}{\Delta x} (Q_{i+1}^n - Q_i^n) \right] \\ - \alpha \left[\theta [(1-\phi) \frac{Q_i^{n+1}}{A_i^{n+1}} + \phi \frac{Q_{i+1}^{n+1}}{A_{i+1}^{n+1}}]^2 + (1-\theta) [(1-\phi) \frac{Q_i^n}{A_i^n} + \phi \frac{Q_{i+1}^n}{A_{i+1}^n}]^2 \right] \\ \left[\frac{\theta}{\Delta x} (A_{i+1}^{n+1} - A_i^{n+1}) + \frac{(1-\theta)}{\Delta x} (A_{i+1}^n - A_i^n) \right] \end{aligned}$$

any location along a link at which the cross-section is known and with which hydraulic parameters are associated; a reach is any stretch of channel between two points. Any link always has at least two points, one associated with the node at each end.

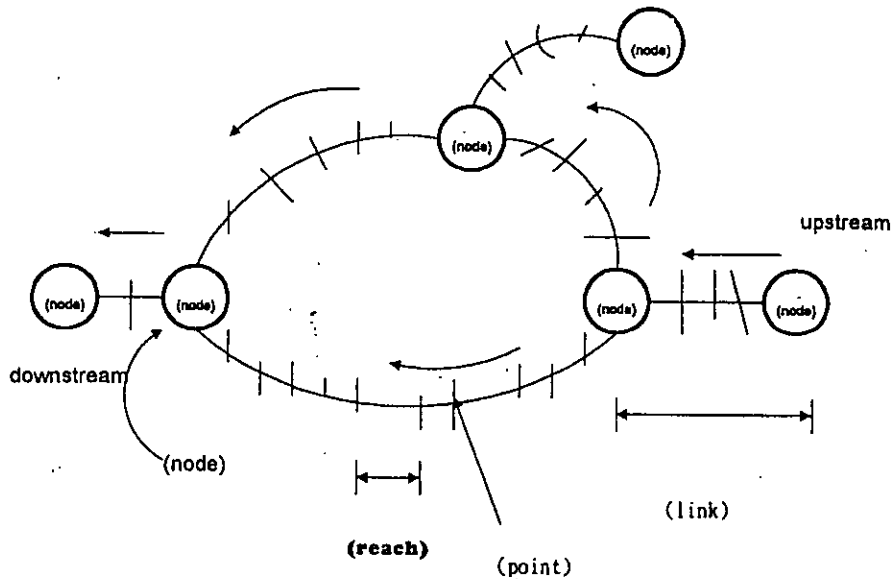


Fig 2.2 Schematic diagram of a portion of a multiply-connected network

The multiply-connected flow paths of a braided river necessitate the use of non-traditional procedures for solving the governing equations. Whereas in a traditional backwater computation the discharge is known a priori, in a multiply-connected system the discharge distribution, i.e., the flow in each channel, must be determined as an integral part of the backwater computation. This determination is based on the application of continuity principles at any junction of two or more channels.

1. Nodal Continuity:

Under the quasi-steady-flow assumption, at any moment the discharge along a link must be constant, i.e., all points must have the same flow. Thus a unique discharge Q_l is associated with each link l at any time. On this basis, the continuity equation (inflow=

and Δy_i is the water-level correction at point $i = I$, the last point on the link.

Coefficients E , F , and H at point $i = I$ for each link cannot be obtained directly, since the hydraulic conditions are not known a priori (except at a boundary point). However, the recursion relation can always proceed downstream without knowing the conditions at point I , because coefficients E , F , and H at the second point can be obtained directly from Eqs. (2.12), (2.13), and (2.17):

$$E_{I(l)-1} = (c_2 b_1 - c_1 b_2) / (d_1 b_2 - d_2 b_1) \quad (2.24)$$

$$F_{I(l)-1} = (g_2 b_1 - g_1 b_2) / (d_1 b_2 - d_2 b_1) \quad (2.25)$$

$$H_{I(l)-1} = (a_1 b_2 - a_2 b_1) / (d_1 b_2 - d_2 b_1) \quad (2.26)$$

Thus, once E , F , H have been initialized by Eqs. (2.24)~(2.26), the remaining E_i , F_i , H_i coefficients can be calculated by recurrence using Eqs. (2.21)~(2.23) for $i = I(l)-1, \dots, 2$. In particular, once E_i , F_i , H_i for $i = 2$ are known, Eq. (2.20) can be written for $i = 1$ as:

$$\Delta Q_1 = E_1 \Delta y_1 + F_1 + H_1 \Delta y_{I(l)} \quad (2.27)$$

Now the same procedures are needed to find the nodal relation for point I . Again, suppose

$$\Delta Q_{I(l)} = E'_{i+1} \Delta y_{i+1} + F'_{i+1} + H'_{i+1} \Delta y_{I(l)} \quad (2.28)$$

From Eqs. (2.12), (2.13), (2.20), and (2.28), the following recursion relations can be obtained:

$$E'_i = E'_{i+1} (c_1 M_{i+1} - d_1 L_{i+1}) / [M_{i+1} (a_1 + b_1 E_{i+1}) - d_1] \quad (2.29)$$

$$F'_i = E'_{i+1} [M_{i+1} (g_1 - b_1 F_{i+1}) - d_1 N_{i+1}] / [M_{i+1} (a_1 + b_1 E_{i+1}) - d_1] \quad (2.30)$$

$$H'_i = -b_1 H_{i+1} E_{i+1} M_{i+1} / [M_{i+1} (a_1 + b_1 E_{i+1}) - d_1] \quad (2.31)$$

Now for $i = I(l)-1$,

$$E'_{I(l)-1} = (c_1 d_2 - c_2 b_1) / (b_1 d_2 - b_2 d_1) \quad (2.32)$$

$$F'_{I(l)-1} = (g_1 d_2 - g_2 d_1) / (b_1 d_2 - b_2 d_1) \quad (2.33)$$

$$H'_{I(l)-1} = (a_1 d_2 - a_2 d_1) / (b_1 d_2 - b_2 d_1) \quad (2.34)$$

Therefore,

This procedure can be expressed as

$$\{\Delta y\} = [A]^{-1}\{B\} \quad (2.38)$$

The block tri-diagonal matrix solution technique is adopted here. The method as described below closely parallels that described in Mahmood and Yevjevich (1975). The basic goal of this technique is to replace the inversion of a NODE×NODE matrix by the inversion of NG matrices, each of size MAXG×MAXG, where NG is the total number of node groups, and MAXG is the maximum number of nodes in a node group.

By definition, a node group is a group of nodes which contains nodes which are linked only to each other, or to nodes of the previous group, or to nodes of the following group. In the following derivations the subscript ng denotes the node group number, $1 < ng < NG$.

The node continuity equation for a node group which is neither the first, nor the last, can be written:

$$[R]_{ng} \{\Delta y\}_{ng-1} + [S]_{ng} \{\Delta y\}_{ng} + [T]_{ng} \{\Delta y\}_{ng+1} = \{V\}_{ng} \quad (2.39)$$

where $\{\Delta y\}_{ng}$ denotes the vector of nodal water level corrections in node group ng, etc., matrices $[R]$, $[S]$, $[T]$ can be thought of as sub-matrices of $[A]$ in Eq. (2.36), and the vector $\{V\}$ can be thought of as a sub-vector of $\{B\}$ in Eq. (2.36).

In order to develop an algorithm which requires inversion of matrices having square dimensions no larger than the number of nodes in the largest node group, one first proposes a relation of the form:

$$\{\Delta y\}_{ng-1} = [E]_{ng-1} \{\Delta y\}_{ng} + \{F\}_{ng-1} \quad (2.40)$$

where $[E]_{ng-1}$ is an (unknown) matrix having NGS_{ng} columns and NGS_{ng-1} rows, and $\{F\}_{ng-1}$ is an (unknown) vector having NGS_{ng-1} rows. If Eq. (2.40) is substituted into Eq. (2.39), the resulting expression becomes a relationship between $\{\Delta y\}_{ng}$ and $\{\Delta y\}_{ng+1}$ which can be written:

$$\{\Delta y\}_{ng} = [E]_{ng} \{\Delta y\}_{ng+1} + \{F\}_{ng} \quad (2.41)$$

Therefore, if $[E]_{ng-1}$ and $\{F\}_{ng-1}$ are known, they can be used with the always known matrices of Eq. (2.39) to compute $[E]_{ng}$ and $\{F\}_{ng}$, for $ng = 2, 3, \dots, NG$. Consequently it is possible to compute and store $[E]_{ng}$ and $\{F\}_{ng}$, for $ng = 1, 2, \dots, NG$.

water levels becomes smaller than a specified value ε . As the time steps increase, the specified criterion ε itself becomes smaller and smaller. In addition, if there is danger of the flow passing locally and temporarily into supercritical regime during the stabilization phase, the convective acceleration terms in the de St. Venant equations can be suppressed for a preliminary volume stabilization; then the process can be repeated retaining the convective term to let the water-surface slope adjust to differences in velocity from one section to another.

Through test experience, the systematic time-step variations and the corresponding specified criterion ε used to control the simulation (i.e., when the maximum water-level change is less than ε the simulation proceeds to the next iteration with the larger time step), have been established as shown in Table 2.1.

Table 2.1 Relationship between Δt and ε

Δt	$0.2t_b$	$0.5t_b$	t_b	$2t_b$	$5t_b$	$10t_b$	$20t_b$	$30t_b$	$40t_b$	$50t_b$
ε	$50\varepsilon_b$	$50\varepsilon_b$	$50\varepsilon_b$	$15\varepsilon_b$	$7\varepsilon_b$	$5\varepsilon_b$	$4\varepsilon_b$	$3\varepsilon_b$	$2\varepsilon_b$	ε_b

In Table 2.1, t_b is a specified fundamental time step which is determined on the basis of the given initial condition (input variable FDELTB). If the initial condition is close to the true steady state condition, t_b can be relatively large, otherwise it must be relatively small. ε_b is the specified fundamental value for the water level change, usually 0.01 feet. As shown in Table 2.1, the time step Δt is maintained until the maximum water-level change between two successive cycles is less than ε , then the next larger time step is adopted, and so on. At the end of this procedure the flow is fully stabilized.

2.3 STEADY FLOW COMPUTATION

2.3.1 Governing Equations

Energy Equation for Steady Water Flow :

$$\frac{d}{dx} \left(z + d + \frac{Q^2}{2gA^2} \right) = -S_f \quad (2.42)$$

2.3.2 Discretization of Equations

The multiply-connected flow paths of a braided river necessitate the use of

$$p_i \Delta y_i + q_i \Delta y_{i+1} + r_i \Delta Q_\ell + s_i = 0 \quad (2.47)$$

In the multiply-connected networks, Eq. (2.47) contains three unknowns Δy_i , Δy_{i+1} , and ΔQ_ℓ . If a link ℓ has $I(\ell)$ total points and $I(\ell)-1$ computational reaches, then there are available $I(\ell)-1$ Eqs. (2.47) and $I(\ell) + 1$ unknown. Thus it is clear that additional equations, arising from imposition of the node continuity Eq. (II.11), must be combined with Eq. (2.47) to obtain a solution for the discharge and level corrections.

One proceeds by seeking to relate the discharge correction in one link, ΔQ_ℓ , to the water-level changes at the nodes situated each end of the link. A procedure analogous to the looped-network double-sweep algorithm described by Cunge et al (1980) involves first hypothesizing the relation :

$$\Delta Q_\ell = E_{i-1} \Delta y_i + F_{i-1} + H_{i-1} \Delta y_i \quad (2.48)$$

Eq. (2.48) can be written for $i = I(\ell)$ as :

$$\Delta Q_\ell = E_{I(\ell)-1} \Delta y_1 + F_{I(\ell)-1} + H_{I(\ell)-1} \Delta y_{I(\ell)} \quad (2.49)$$

In the node-continuity Eq. (2.15), each link-discharge correction ΔQ_m , ℓ can now be replaced by Eq. (2.49) for the appropriate link, transforming it to :

$$\begin{aligned} Q_m(t_{n+1}) + \sum_{\ell=1}^{L(m)} Q_{m,\ell} + \sum_{\ell=1}^{L(m)} (E_{I(\ell)-1,t} \Delta y_{1,t} + F_{I(\ell)-1,t} + H_{I(\ell)-1,t} \Delta y_{I(\ell),t}) \\ = 0, m = 1, 2, \dots, M \end{aligned} \quad (2.50)$$

At this point an important additional constraint is introduced, namely that all points associated with a node m share the same common water level correction, Δy_m .

$$[A] \{ \Delta y \} = \{ B \} \quad (2.51)$$

where $\{ \Delta y \}$ is the vector of water-level corrections at the nodes, Δy_m , $m = 1, 2, \dots, M$, $[A]$ is the coefficient matrix comprising appropriate summations of $E_{I(\ell)-1}$ and $H_{I(\ell)-1}$ coefficients, and B is a known vector whose elements are imposed inflows, and sums of latest discharge estimates and $F_{I(\ell)-1}$ coefficients.

2.3.3 Solution Strategy

2.4 BACK WATER COMPUTATION (I)

Energy equation for non-prismatic channels takes the form in the trial-and-error step method:

$$z_1 + y_1 + \alpha_1 \frac{Q_1^2}{2gA_1^2} = z_2 + y_2 + \alpha_2 \frac{Q_2^2}{2gA_2^2} + h_f + h_e \dots \quad (2.54)$$

where subscript 1 represents upstream section and subscript 2 represents downstream section,; z = bed elevation at cross section; α_1 = upstream velocity correction coefficient (momentum coefficient).

$$h_f = \text{frictional losses} = S_f \Delta x = \frac{(S_{f1} + S_{f2}) \Delta x}{2} \dots (2.55)$$

$$h_e = \text{energy losses due to eddies} = k \Delta \left(\alpha \frac{Q^2}{2gA^2} \right) \dots (2.56)$$

in which k is the energy loss coefficient due to eddies

Eq. (2-54) may be rewritten as

$$z_1 + y_1 + \alpha_1 \frac{Q_1^2}{2gA_1^2} = z_2 + y_2 + \alpha_2 \frac{Q_2^2}{2gA_2^2} + \frac{(S_{f1} + S_{f2}) \Delta x}{2} + k \Delta \left(\alpha \frac{Q^2}{2gA^2} \right) \dots (2.57)$$

and define

$$H_1 = z_1 + y_1 + \alpha_1 \frac{Q_1^2}{2gA_1^2} \dots (2.58)$$

$$H_2 = z_2 + y_2 + \alpha_2 \frac{Q_2^2}{2gA_2^2} \dots (2.59)$$

then

$$H_1 = H_2 + h_f + h_e \dots (2.60)$$

For subcritical flow, we may assume downstream (subscript 2) is given and perform computation towards upstream. Trial-and-errors method assumes upstream stage equals downstream stage initially and revises upstream value each step till H_1 values computed by (2.58) and (2.60), respectively, coincide. This method uses trial-and-error to revise H_1 value until the deviation previous H_1 value approaches to zero. Let e be the deviation in H_1 , then

$$\frac{de}{dy_1} = \frac{d}{dy_1} \left(y_1 + \alpha_1 \frac{Q_1^2}{2gA_1^2} - \frac{1}{2} \Delta x S_{f1} \right) \dots (2.61)$$

node, and \bar{y} be the average stage of all links on downstream side of the node. Assume Δy_i to be stage correction and ΔQ_i to be discharge correction for the i th link, the following relations may be established :

$$Q = AV = A \frac{1}{n} R^{2/3} S^{1/2} \dots \dots \dots (2.64)$$

$$\Delta Q_i = \frac{\partial Q}{\partial y} \Delta y_i = \left(\frac{2}{3} \frac{1}{R} \frac{\partial R}{\partial y} Q + \frac{\partial Q}{\partial y} \frac{Q}{A} \right) \Delta y_i \dots \dots \dots (2.65)$$

where $\Delta y_i = \bar{y} - y_i$ and $\bar{y} = \frac{y_1 + y_2 + \dots + y_n}{n}$.

(2) The sum of all discharge corrections ΔQ_i must be zero. If $\Delta Q_1 + \Delta Q_2 + \dots + \Delta Q_n$

$= R_n \neq 0$, then $\Delta Q_i = \Delta Q_i - \frac{Q_i}{\sum_{j=1}^n Q_j} R_n$ are taken as new discharge corrections for next

computation . This procedure is repeated until stages of all links are approximately equal (error of 0.01 m as default value) or the number of iterations reaches 50. In the latter case, the solution for stage and discharge with least error will be selected. The approach may also be used for computing initial stage and discharge.

(3) Generally, allocation of link discharges at a node is considered in proportion to the discharges resulted from previous time step. If upstream flows and/or lateral flows change at any time step, data-tree search technique is employed to re-calculate discharge allocation and corrections for stage and discharge until the sum of discharges and deviations of stages at the node approach zero. This technique of approach extends present backwater computation to river network solutions for stage and discharge, and may be time-consuming as compared to total solution approach.

2.5 BACK WATER COMPUTATION (II)

For most of the water profile computations, NETSTARS uses the energy equations

$$z_1 + y_1 + \alpha_1 \frac{Q_1^2}{2gA_1^2} = z_2 + y_2 + \alpha_2 \frac{Q_2^2}{2gA_2^2} + h_f \quad (2.66)$$

where z = bed elevation; y = water depth; α = velocity distribution coefficient; h_f = total energy loss between section 1 and 2; and subscripts 1 and 2 denotes sections 1 and 2, respectively.

where $SF(d)$ = specific force corresponding to a water depth d ; A_t = total flow area; and A_m = flow area in which motion exists.

The sequent depth is computed where hydraulic jumps occur. An iterative trial-and error procedure is used to find the sequent water surface elevation. The process starts with two guesses; the critical water surface elevation with the theoretical minimum specific force, and the maximum bottom elevation for the cross section. The subcritical sequent water surface elevation is located within these two values. The bisection method is used to solve equation

$$SF(d_a) = SF(d_b) \quad (2.72)$$

where d_a = computed supercritical water surface elevation, and d_b = desired subcritical sequent water surface elevation.

Detailed procedures for normal, critical, and sequent depth computations can be found in open channel hydraulics books (e.g., Chow, 1959; Henderson, 1966) and in the paper by Molinas and Yang (1985).

2.6 GEOMETRIC COMPUTATIONS

The river reach to be modeled must be described by a finite number of discretized cross sections. Cross section geometry is described by X-Y coordinate pairs, i.e., by coordinate pairs with lateral location and bed elevation.

For natural channels of irregular cross section, the channel can be divided into subchannels. The variables related to the cross-sectional geometry (area, wetted perimeter, hydraulic radius, channel's top width) are computed for each subchannel. These values are summed to obtain the total values for the cross section. The relationships used are well known in the literature and are the following:

$$A_i = 0.5(y_i + y_{i+1})dx_i \quad (2.73)$$

$$P_i = [dx_i^2 + (y_i - y_{i+1})^2]^{1/2} \quad (2.74)$$

$$R_i = \frac{A_i}{P_i} \quad (2.75)$$

$$\bar{y}_i = 0.25(y_i + y_{i+1}) \quad \text{if not adjacent to channel wall} \quad (2.76)$$

NETSTARS model.

$$EFD = \frac{\sum_{i=1}^{i_t} D_{avg} a_i D_{avg}^{2/3}}{\sum_{i=1}^{i_t} a_i D_{avg}^{2/3}} \dots \dots \dots (2.84)$$

where D_{avg} is the average depth of trapezoidal area below water surface, a_i is the water area of subdivided cross section, i_t is the total number of trapezoids in subdivided cross section. Bed elevations are expressed in four different ways in the NETSTARS, i.e., (1) minimum bed elevation, (2) average bed elevation = water surface elevation – EFD, (3) average bed elevation = water surface elevation – hydraulic depth R_h , where $R_h = \frac{A_t}{T_t}$,

$$A_t = \sum_{j=1}^{Nstube} A_j, T_t = \sum_{j=1}^{Nstube} T_j, T_t \text{ is top width, (4) average bed elevation = average elevation of}$$

the cross section in which distance between two neighboring cross sectional points is used a weighting factor. The former three expressions indicate that bed elevation is always below the water stage. Expression (1) shows only minimum bed elevation to emphasize local change, and does not reflect scour and deposition characteristics of the entire cross section. For irregular channels where main channel and its overbanks are clearly distinguishable or thalweg is well defined, expression (1) is deemed adequate to reflect scour and deposition characteristics of the bed. When overbanks are not distinguishable from main channel and cross sections of the channel are extremely irregular, expression (4) seems to better reflect average scour and deposition behavior of channel bed. The fact that bed elevation is higher than water stage would generally with main channel is distinguishable from its overbanks or thalweg in well defined in channel. In this case, it is better to use the first three expressions for bed elevation. Expressions (2) and (3) refer to water surface elevation, hence the water surface elevation must be maintained at same level for comparison of average scour and deposition characteristics.

2.7 HYDRAULIC PARAMETERS FOR SEDIMENT COMPUTATION

Hydraulic parameters required for sediment load computation are flow velocity, flow depth, hydraulic radius, top width, friction slope and flow discharge at the cross section.

2.8 ROUGHNESS COEFFICIENTS

3. SEDIMENT ROUTING

3.1 INTRODUCTION

Transport of sediments in natural streams may be divided into bed load, suspended load and wash load according to sediment moving behaviors. The combination of bed load and suspended load is called bed material load of which the stream bed is composed. The wash load is composed of sediment particles of sizes finer than those represented in the bed and stays in suspended state all the time. For scour and deposition study of river bed, the bed material load is considered as the total load.

Sediment routing is performed with given stages and discharges, obtained by flow routing, for each stream tube in the cross section per stream tube concept. Sediment loads are computed for each stream tubes of equal conveyance in the cross section (per stream tube concept) and continuity equation of sediments is applied to determine average bed change in each stream tube. Since the discharge and bed form may change from time step to time step, boundaries of subdivided stream tubes may vary accordingly, as a result, transverse bed form change is simulated.

Sediment load computation, bed sorting and armoring, sediment load conditions at boundaries and nodes, and sediment size demarcation for bed load and suspended load are described in the following sections.

3.2 STREAM TUBE CONCEPT

By definition, a streamline is a conceptual line to which the velocity vector of the fluid is tangent at each and every point, at each instant in time. Stream tubes are conceptual tubes whose walls are defined by streamlines. The discharge of water is constant along a stream tube because no fluid can cross the stream tube boundaries. Therefore, the variation of the velocity along a stream tube is inversely proportional to the stream tube area. Fig. 3.1 illustrates the basic concept of stream tubes used in NETSTARS.

NETSTARS uses the stream tube concept to accomplish a semi-two-dimensional approximation of the region being modeled. This allows the program to consider not only longitudinal, but also lateral variations of the hydraulics and sediment activity at each cross section of the study. The use of stream tubes by NETSTARS is described in this section.

The water surface profiles are computed first, as described in the previous sections of this

separately for each stream tube. In NETSTARS, lateral variations of bed material composition are accounted for, and this variation is included in the computations of the bed material composition and sorting for each stream tube. Therefore, although no material is allowed to cross stream tube boundaries during a time step, lateral movement of sediment is accomplished by the lateral variation of the stream tube boundaries from time step to time step.

NETSTARS is not a truly two-dimensional program, therefore it cannot simulate areas with recirculating flows or eddies. Other limitations include the inability of simulate secondary flows, reverse flows or eddies. Other limitations include the inability of simulate secondary flows, reverse flows, water surface variations in the transverse direction, hydrograph attenuation, and others that result from the use of the simplified governing equations described in this chapter.

3.3 TOTAL LOAD COMPUTATION

NETSTARS model provides two options to estimate total load of sediments. One is the direct estimate of bed-material load, and the other takes summation of separate estimates of bed load and suspended load. For suspended-load-controlled river reaches, the later approach is performed.

3.3.1 Sediment Transport Functions

NETSTARS model provides four different formulas to compute total load : (1) Yang's (1973,1984), (2) Ackers-White (1973), (3) Engelund and Hansen (1967), and (4) Van Rijn (1984,1985). The computed result from either formula is expressed in sediment concentration by weight of the mixture, C_s . If the sediments are non-uniform, $C_s = \sum_{i=1}^n P_i C_{si}$ in which P_i is % by weight of the i th size fraction. The total load Q_s is determined by

$$Q_s = \gamma \int_{B_L}^{B_R} \int_a^D C_s u d_x d_B \quad (3.1)$$

where u = average flow velocity (L/T); C_s = sediment concentration by weight (M/M); Q_s = total load ($\frac{ML}{T^3}$); $\gamma = \rho g$, unit weight of sediment ($\frac{M}{L^2 T^2}$); B = coordinate system for cross sediment (L); B_L = left boundary of stream tube (L); B_R = right boundary of stream tube (L); D = flow depth (L); a = bed layer thickness (L).

$$\Delta Z_{i,k} = \frac{\Delta t}{(1-p_i)} \cdot \frac{q_{lat}(\Delta x_i + \Delta x_{i-1}) + 2(Q_{s,i-1,k} - Q_{s,i,k})}{(aT_{i-1} + bT_i + cT_{i+1})(\Delta x_i + \Delta x_{i-1})} \quad (3.6)$$

where κ = size fraction index; p_i = porosity at cross section i ; and $Q_{s,i,k}$ = computed volumetric sediment discharge for size class κ at cross section i . The total bed elevation change for a stream tube at cross section i , ΔZ_i , is computed from

$$\Delta Z_i = \sum_{k=1}^N \Delta Z_{i,k} \quad (3.7)$$

where N = total number of size fractions present in cross section i . The new channel cross section at station i , to be used at the next time iteration, is determined by adding the bed elevation change to the old bed elevation.

3.4 BED LOAD AND SUSPENDED LOAD COMPUTATION

In a model bed load and suspended load are not treated separately, hence it can not reflect the non-equilibrium deposition of the suspended sediment. To remedy the shortcomings of this type of models, an alternate approach is developed herein. This approach uses the stream tube concept and includes the capability of simulating the movement of suspended load and bed load, and their interactions. Hence, it is able to simulate deposition patterns of the suspended sediment in a non-equilibrium process.

3.4.1 Sediment Continuity Equation

3.4.1.1 Governing Equation

The sediment continuity equation is given as

$$(1-p) \frac{\partial A_d}{\partial t} + \sum_{k=1}^{Nsize} S_k + \frac{\partial Q_b}{\partial x} = q_{sl} \quad (3.8)$$

Eq. (3.8) can be rewritten as

$$(1-p) \frac{\partial A_d}{\partial t} + \frac{\partial}{\partial x} \sum_{k=1}^{Nsize} [q_t C_k] + \frac{\partial Q_b}{\partial x} = q_{sl} \quad (3.9)$$

where Q_b = bed load transport rate in stream tube; q_t = flow discharge in stream tube; and C_k = depth-averaged concentration of suspended sediment of size fraction k in stream tube.

$$S_{ek} = \rho B_t W_k \beta_k C_{ek} \quad (3.14)$$

where B_t = width of stream tube; ρ = sediment-water mixture density; W_k = fall velocity of sediment of size fraction k ; β_k = weight percentage of sediment of size fraction k ; and C_{ek} = sediment concentration close to channel bed, which can be calculated by the equation proposed by Van Rijn (1984)

$$C_{ek} = 0.015 \frac{D_k T_k^{1.5}}{a D_*^{0.3}} \quad (3.15)$$

where D_k = particle diameter of size fraction k ; D_* = particle parameter = $D_{50} \{[(s-1)g]/\nu^2\}^{1/3}$; T_k = transport stage parameter = $[(u'_*)^2 - (u_{*cr})^2] / (u_{*cr})^2$; ν = water kinematic viscosity; s = specific weight of sediment particle; u'_* = grain shear velocity = $g^{0.5} u / c'$; c' = Chezy coefficient related to grains = $18 \log(12 R_t / 3D_{90})$; R_t = hydraulic radius in stream tube; and u_{*cr} = critical shear velocity.

The amount of sediment deposition can be calculated by using the following equation (Holly and Rahuel 1990):

$$S_{dk} = -\rho B_t W_k C_{dk} \quad (3.16)$$

where C_{dk} = deposition concentration, which can be estimated by

$$C_{dk} = [3.25 + 0.55 (\ln(W_k / k u_*))] C_k \quad (3.17)$$

with $k = 0.4$.

3.4.3.2 Discretization of Equation

The concentration C_k is obtained by solving the convection diffusion equation, i.e., Eq. (3.12). The split operator approach is used in solving this equation. The governing equation is separated into four portions, i.e., advection, longitudinal diffusion, transverse diffusion, and reaction. They are solved subsequently in one time step. The C_k and CX_k , $CX_k = \partial C_k / \partial x$, values obtained in the previous portion are served as the known values for the next portion. The computational techniques are described as the following: (To simplify the expression C is used to replace C_k from here on.)

(1) Advection Step.

$$B_3 = (U_i^n / U_i^t) s' (3s' - 2); \text{ and } B_4 = (U_i^{n+1} / U_i^t) (s' - 1) (3s' - 1).$$

(2) Longitudinal Diffusion Step

The longitudinal diffusion portion of Eq. (3.11) can be written as

$$\frac{\partial C}{\partial t} - \frac{1}{A} \frac{\partial}{\partial x} (AK_x \frac{\partial C}{\partial x}) = 0 \quad (3.23)$$

Using the Crank-Nicholson central difference method, Eq. (3.22) can be discretized as

$$C_i^{n+1} - C_i^n = f_1(C_{i+1}^{n+1} - C_i^{n+1}) - f_2(C_i^{n+1} - C_{i-1}^{n+1}) \quad (3.24)$$

where

$$f_1 = \frac{\Delta t}{\frac{1}{2}(x_{i+1} - x_{i-1})A_i} \frac{[\frac{1}{2}(A_i K_{xi} + A_{i+1} K_{xi+1})]}{(x_{i+1} - x_i)}$$

$$f_2 = \frac{\Delta t}{\frac{1}{2}(x_{i+1} - x_{i-1})A_i} \frac{[\frac{1}{2}(A_i K_{xi} + A_{i-1} K_{xi-1})]}{(x_i - x_{i-1})}$$

Differentiating Eq. (3.23) with respect to x , and then using the numerical scheme, the difference equation becomes

$$CX_i^{n+1} - CX_i^n = g_1 CX_{i+1}^{n+1} - (g_2 + g_3) CX_i^{n+1} + g_4 CX_{i-1}^{n+1} \quad (3.25)$$

Where

$$g_1 = \frac{\Delta t}{\frac{1}{2}(A_{i+1} + A_i)} \frac{A_{i+1} K_{xi+1}}{\frac{1}{2}(x_{i+1} - x_{i-1}) \cdot (x_{i+1} - x_i)}$$

$$g_2 = \frac{\Delta t}{\frac{1}{2}(A_{i+1} + A_i)} \frac{A_i K_{xi}}{\frac{1}{2}(x_{i+1} - x_{i-1}) \cdot (x_{i+1} - x_i)}$$

$$g_3 = \frac{\Delta t}{\frac{1}{2}(A_{i-1} + A_i)} \frac{A_i K_{xi}}{\frac{1}{2}(x_{i+1} - x_{i-1}) \cdot (x_i - x_{i-1})}$$

$$g_4 = \frac{\Delta t}{\frac{1}{2}(A_{i-1} + A_i)} \frac{A_{i-1} k_{xi-1}}{\frac{1}{2}(x_{i+1} - x_{i-1}) \cdot (x_i - x_{i-1})}$$

The values of C and CX can be obtained by using Gaussian Elimination Method to

where $a_p = a/A$ and $b_p = b/A$. There exists an analytical solution for Eq. (3.29), and shown as

$$C_i^{n+1} = (a_p / b_p) + (C_i^n - a_p / b_p) e^{-b_p \Delta t} \quad (3.30)$$

Differentiating Eq. (3.29) with respect to x and the difference expression form for CX_i^{n+1} can be obtained as

$$CX_i^{n+1} = \frac{1}{(1 + b_{pi} \Delta t)} [CX_i^n + (\frac{a_{pi} - a_{pi-1}}{\Delta x}) \Delta t - (\frac{b_{pi} - b_{pi-1}}{\Delta x} \cdot C_i^{n+1}) \Delta t] \quad (3.31)$$

3.4.3.3 Solution Strategy

The advection diffusion equation is solved in form separate parts using split operation method. Concentrations terms in advection and in longitudinal diffusion are solved as in a single stream tube within the network. The transverse diffusion is the concentration exchange between stream tubes in a cross section and may be solved Gauss theorem. The concentration terms in reaction are solved as in a single stream tube in a cross section.

Longitudinal and transverse dispersion coefficients are estimated by using Elder's (1959) empirical equations: $k_x = 5.93u_s d$ and $k_z = 0.23u_s d$. These coefficients should be properly adjusted for reaches of tidal effects.

With given conditions, the advection diffusion equation is solved in form separates parts to revise values of C_k and CX_k in stream tubes. Details of solution method are described as follows.

1. Advection

(1) Boundary conditions and initial conditions

B.C. and I.C. at both upstream and downstream must be given to solve the advection diffusion equation for C_k values.

(A) C_N^{n+1} known.

(B) $CX_N^{n+1} = CX_{N-1}^{n+1}$ indicating long downstream reach with open boundary; a national control section on the stream; advection of suspended sediments exists.

(C) $CX_N^{n+1} = 0$ indicating closed boundary, no advection occurs. Initial value of CX_k may be obtained from the variation of C_k with respect to x . However, boundary condition of CX_k is difficult to attain. Fortunately, the accuracy of initial and

(D) After computations of all links upstream of the node are completed in step (C), C_k and CX_k values at the first computational point of downstream links from the node may be determined using Eqs. (3-32) and (3-33).

(E) Steps (C) and (D) are repeated until finished.

2. Longitudinal Diffusion

(1) Boundary conditions and initial conditions

Three types of boundary conditions are:

(A) C_N^{n+1} known.

(B) $CX_N^{n+1} = CX_{N-1}^{n+1}$ indicating long downstream reach with open boundary, a natural control section on the stream, diffusion of suspended sediments exists.

(C) $CX_N^{n+1} = 0$ indicating closed boundary, no diffusion of suspended sediments exist.

Boundary conditions applied depend upon the coefficients of finite difference equations transformed from longitudinal diffusion equation by Tee Scheme. Initial conditions for longitudinal diffusion equation are taken from the resulted C_k and CX_k of advection solution.

(2) Node continuity

The assumptions that no suspended sediments are accumulated at the node and concentration changes though the node due to diffusion remain intact call for

$$\sum K_i A_i CX = 0 \quad (3.34)$$

$$\Delta C_p = \Delta C_q \quad (3.35)$$

where p and q are computational point connected to the node. If the values of concentration gradient of all links at a node CX are computed using node input data or assumed boundary conditions and resulted such that $\sum K_i A_i CX = R_n \neq 0$. A correction of $-(R_n / l) / (k_x A_i)$ for CX must be added to each link so that node continuity, $\sum K_i A_i CX = 0$, is satisfied.

(3) Computation Procedures

(A) Obtain upstream boundary values of C_k^{n+1} and CX_k^{n+1} . If only C_k^{n+1} values are given, assume $CX_k^{n+1} = 0$. If only CX_k^{n+1} values are given, C_k^{n+1} values are computed using C_k^n of next computational point in previous time step.

(B) Values of C_k^{n+1} or CX_k^{n+1} are taken as downstream boundary conditions. Compute

each stream tube should be converted to volume rate in L^3/T unit.

2. Values of C_k and Q_b from step 1 are substituted into Eq. (3.10) to solve for ΔZ_i . Bed elevations are revised accordingly.
3. Hydraulic computations for stage, discharge and stream tube subdivision are performed in each time step. And then execute steps 1 and 2. The three steps are repeated until the end of simulation time.

3.5 BED SORTING AND ARMORING

Consequently, several different processes may take place. For example, all the finer particles may be eroded, leaving a layer of coarser particles for which there is no carrying capacity. No more erosion may occur for those hydraulic conditions, and the bed is said to be armored. This armor layer prevents the scour of the underlying materials and the sediment available for transport becomes limited to the amount of sediment entering the reach. However, future hydraulic events, such as an increase of flow velocity, may increase the flow carrying capacity, causing the armor layer to break and restart the erosion processes in the reach.

Many different processes may occur simultaneously within the same channel reach. These depend not only on the composition of the supplied sediment, i.e., the sediment entering the reach, but also on bed composition within that reach. The bed composition may vary within the reach both in space and time. In order to model these type of events, GSTARS 3.0 uses the bed composition accounting procedure proposed by Bennett and Nordin (1977).

In Bennett and Nordin's method, bed accounting is accomplished by the use of two or three conceptual layers (three layers for deposition and two layers for scour). The process is schematically illustrated in Fig. 3.2. The top layer, which contains the bed material available for transport, is called the active layer. Beneath the active layer is the inactive layer, which is the layer used for storage. Below these two layers there is the undisturbed bed, with the initial bed material composition.

The active layer is the most important concept in this procedure. It contains all the sediment that is available for transport at each time step. The thickness of the active layer is defined by the user as proportional to the geometric mean of the largest size class containing at least 1 percent of the bed material at that location. Active layer thickness is, therefore, closely related to the time step duration. Erosion of a particular size class of bed material is limited by the amount of sediments of that size class present in the active layer. If the flow

cross section of the link), continuity of sediment loads at the node requires that

$$Q_s^{n+1} + \sum_{i=1}^{L(m)} Q_{s_{in,m,j}}^{n+1} = \sum_{j=1}^{L(m)} Q_{s_{out,m,j}}^{n+1} \quad (3.36)$$

$$Q_{s_{out,m,j}}^{n+1} = \frac{Q_{out,j}}{\sum_{k=1}^{L(m)} Q_{out,k}} \left(Q_s^{n+1} + \sum_{j=1}^{L(m)} Q_{s_{in,m,j}}^{n+1} \right) \quad (3.37)$$

where $L(m)$ = total number of links connected to node m ; m = total number of nodes;

Q_s^{n+1} = external sediment load added to node m at time step $n+1$ (L^3/T); $Q_{s_{out,m,j}}^{n+1}$ = sediment load of the i th link at nose m (L^3/T).

(B) In proportion to sediment transport capacity

The following continuity relationship of sediments at a node may be established using the rating curve of sediment transport capacity versus discharge ($Q_s = A_c Q^{B_c}$) at the first cross section of each link downstream from the node.

$$Q_s^{n+1} + \sum_{i=1}^{L(m)} Q_{s_{in,m,j}}^{n+1} = \sum_{j=1}^{L(m)} Q_{s_{out,m,j}}^{n+1} \quad (3.38)$$

$$Q_{s_{out,m,j}}^{n+1} = \frac{A_{c,j} Q_{out,j}^{B_{c,j}}}{\sum_{k=1}^{L(m)} (A_{c,j} Q_{out,k}^{B_{c,j}})} \left(Q_s^{n+1} + \sum_{j=1}^{L(m)} Q_{s_{in,m,j}}^{n+1} \right) \quad m = 1, 2, \dots, M \quad (3.39)$$

where $A_{c,j}$ = coefficient in the relationship between sediment transport capacity and flow discharge; $B_{c,j}$ = exponent in the relationship between sediment transport capacity and flow discharge.

The sediment load allocated to each link downstream from the node m is directly proportional to the link sediment transport capacity as computed from the relationship of sediment transport capacity versus flow discharge at the first cross section of the link downstream from the node.

2. Boundary conditions for sediment loads

(A) Sediment load time series

As obtained from field measurements of sediment loads.

(B) Sediment rating curves

As obtained from statistical analysis of long-term records of flow discharge and

4. DATA REQUIREMENTS

This chapter describes the hydraulic and sediment data requirements for NETSTARS. Moreover, this chapter also describes the overall data needs and some of the formats used to input data.

4.1 INPUT CONTROL

4.1.1 Channel Geometry Data

Channel geometry data include cross section geometry, location, and channel roughness. Data are entered in records. Record BR contains the location, elevation of lowest point in the initial cross section, and channel roughness. Record NS is used to specify number of data points in the cross section, increment in bed elevation, enlargement coefficient for the width, scorn limits at river banks and bed elevation. Record GR is used to define the cross section geometry at the given station using X-Y coordinate pairs, i.e., by coordinate pairs with lateral location (X) and bed elevation (Y).

4.1.2 Hydraulic and Hydrologic Data

Hydraulic and Hydrologic data include water discharge, stage, and rainfall. Record AB selects the hydraulic model to execute the flow routing. Record BR contains the initial estimate of water surface elevation and water discharge. Record RE is used to select the calculation method for friction loss calculations and the roughness equation. Record CQ contains water discharge and stage hydrograph at the given node. Record TT is used to define a title.

4.1.3 Sediment Data

Sediment data includes bed material size distributions for the reach of study, the sediment inflow hydrograph entering the reach, stream tube, and sediment transport function. Record AB selects the sediment inflow type at the upstream boundary. Record BC contains the sediment discharge entering the study reach at the cross section farthest upstream as a function of the water discharge. Record BS contains the number of size fractions associated with this cross section. Record CQ contains sediment discharge at the given node. Record IT contains the desired number of time steps for the sediment-routing procedures. Record NT contains the number of stream tube used in sediment-routing computations. Record SE

Record AC is used to specify method to determine water stage and suspended sediment concentration gradient on the downstream boundary. Record AJ is used to specify grouping of nodes. Record BC is used to specify the rating curve of sediment load versus flow discharge for sediment inflow computation at upstream boundary. Record BK is used to specify links at a node. Record BL is used to specify link including nodes and computation point on the link. Record CC is used to specify suspended sediment concentration time series at a node. Record CQ is used to specify time series of hydraulic and sediment load at a node. Record SN is used to specify size fraction of sediment at a node.

4.1.7 Other Special Data

Record AB is used to specify time step for each step in concentration. Record AI is used to specify number of changes in time step, units (metric or English system) in computation, dispersion coefficients determination, inclusion of lateral flows, and output of longitudinal bed profile. Record AM is used to specify maximum number of iterations in hydraulic computation, roughness coefficient determination, include of sediment computation. Record AT is used to specify beginning and end times of simulation, and time step Δt . Record BE is used to specify error tolerance of water stage and discharge in iteration of hydraulic computations. Record BP is used to specify specific gravity and porosity of sediment, time and space weighty factors, θ and Φ , in de Saint Venant equation, and initial slope of riverbed for steady-flow computation. Record DT is used to specify the time when time step change is required, and the value for new time step.

4.2 OUPUT CONTROL

4.2.1 Initial Data

Record AP allows the user to select link and node data, starting cross section at computation point and sediment load in each stream tube to be printed out. Most of the input data will be echoed in NETS03.OUT file for data verification.

4.2.2 Flow Data

Record PR provides four options for printouts of hydraulic computations, i.e., (1) no printout required, (2) printout for water surface profile only, (3) add normal and critical depth table to printout, and (4) print all.

5 INPUT DATA RECORDS

5.1 INPUT DATA FORMAT

In NETSTARS the data is tabulated in ASCII files. The file is organized in sequential records. A record is a line of up to 80 characters in length that is divided into fields of fixed width (see Fig. 5.1). Fields are numbered from left to right, starting in the left-most character. Field 0 is 2 characters long and is used to specify the record name (all record names are 2 characters long). Fields 1 to 10 are used to input data to NETSTARS. Field 1 is 6 characters long; fields 2 to 10 are 8 characters long.

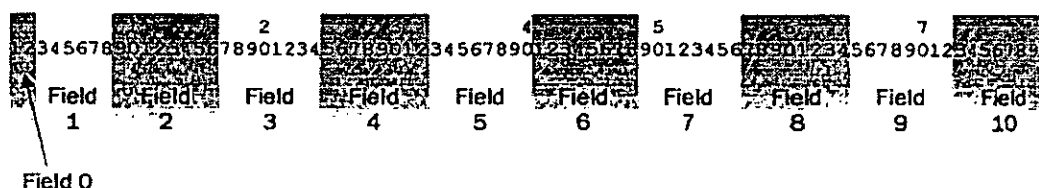


Fig. 5.1 Organization of a data record into different fields.

Each record name is unique and is used to input specific data to the program. A comprehensive list of all the records used by NETSTARS is given in C.2. Not all records are used (for example, some are mutually exclusive) but they have to be in an appropriate sequence. The data requirements presented in this chapter follow the order that should be used when preparing data input for NETSTARS.

5.2 LIST OF INPUT DATA RECORDS

List of input data records used by NETSTARS as follow:

- 1.Record AB Used to specify time unit and methods to compute incoming sediment load and hydraulic parameters.
- 2.Record AC Used to specify methods to determine water surface elevation suspended load concentration at downstream boundary.
- 3.Record AI Used to specify output options and methods to compute sediment transport parameters.

24.Record	IT	Used to specify the execution number of computations in each time step for sediment routing.
25.Record	LA	Used to specify number of cross sections with lateral inflows.
26.Record	LI	Used to specify locations of cross sections with lateral inflows.
27.Record	LT	Used to specify the title for lateral inflows data file.
28.Record	NS	Used to specify cross section data for scour and deposition.
29.Record	NT	Used to specify number of stream tubes in a link.
30.Record	PR	Used to specify printout for computational results of water surface elevations and other hydraulic parameters.
31.Record	RI	Used to specify time series for lateral inflow.
32.Record	SE	Used to specify methods for sediment load calculations and maximum scour depth.
33.Record	SF	Used to specify number of grain size fraction for the sediments.
34.Record	SG	Used to specify end values of each fraction of grain sizes.
35.Record	SN	Used to specify percent of sediment smaller than the indicated size fraction per sieve analysis, i.e., cumulated grain size distribution data at a node.
36.Record	SP	Used to specify percent of sediment smaller than the indicated size fraction per sieve analysis, i.e., cumulated grain size distribution data at a cross section.
37.Record	ST	Used to specify the title for sediment transport data file.
38.Record	TO	Used to specify the time when computational results at that specified time are to be stored in output data file.
39.Record	TT	Used to specify the title for hydraulic data file.
40.Record	X3	Used to specify floodplain area in relation to main channel.

<u>File</u>	<u>Variable</u>	<u>Value</u>	<u>Description</u>
		=2	Use energy equation for steady flows to compute water surface profiles. (Double sweep method)
		=3	Backwater computation (I) (Non-prismatic channel)
		=4	Backwater computation (II) (Capable of hydraulic jump computation)
4	ICARD	=1	Read in data from record X3.
		≠1	No data read from record X3.
5	ICONT	=1	Use hydraulic computation (II) to obtain starting water surface elevation.
		=2	Use hydraulic computation (I) to obtain starting water surface elevation.
		≠1,2	No backwater computation required.

<u>File</u>	<u>Variable</u>	<u>Value</u>	<u>Description</u>
		=3	For long river reach (link), C and $\frac{\partial c}{\partial x}$ assume the resulted values of previous time step. Starting value for $\frac{\partial c}{\partial x}$ is assumed to be 0.
3	INODES		Allocation of sediment loads at nodes.
		=0	In proportion to discharge values.
		≠0	Use rating curves of sediment load versus flow discharge.
4	IEQSED		Selection of continuity equation for sediment transport.
		=0	Use Eq. (3.8)
		≠0	Use Eq. (3.7) with S_k replacing suspended load term.
5	IRESV		Suspended sediment computations.
		=0	High flow discharges in natural streams.
		≠0	Reservoir computation or moderate/low flow discharges.

<u>Field</u>	<u>Variable</u>	<u>Value</u>	<u>Description</u>
		7	(1)(4)
		8	(2)(3)
		9	(2)(4)
		10	(3)(4)
		11	(1)(2)(3)
		12	(1)(2)(4)
		13	(1)(3)(4)
		14	(2)(3)(4)
		15	(1)(2)(3)(4)
6	IFXZ	=1	Use empirical equations: Longitudinal dispersion coefficient (k_x) = 5.93 du_* . Transverse dispersion coefficient (k_z) = 0.23 du_* .
		≠1	k_x and k_z from assigned values in Record CF. (0 is the default value.)
7	JLAT	=1	Add lateral flows. Data read in from NETS 88.LAT.
		≠1	No lateral flows
8	JAVGB	0	Output longitudinal profile of lowest bed elevations.
		1	Output longitudinal profile of average bed elevations as obtained from HEC-6 effective depth computations
		2	Output longitudinal profile of average bed elevations as obtained from hydraulic depth computations
		3	Output longitudinal profile of average bed elevations as obtained from computations of distances between adjacent cross sections.
9		0	Use Manning formula
		1	Use Chezy formula
		2	Use Darcy-Weisbach formula
			The roughness coefficients, n, c, f, in above-mentioned formulas are the FFFACT values in Record BR of NETSO5.HYD file. Only Manning's n value can be automatically calculated.

Record AM

Required

<u>Field</u>	<u>Variable</u>	<u>Value</u>	<u>Description</u>
0	ID	AM	Record identification.
1	MITHYD	+	Maximum number of iterations for steady-flow computation.
2	IFFACT		Friction coefficient
		0	Read from input data.
		1	Use Manning Equation.
		2	Use Limerinos' (1970) formula.
3	ISED1	≤ 0	Without sediment computation.
		> 0	Deal with sediment computation.
4	ITERMX	+	Maximum number of Newton-Raphson iterations, default value is 2.
5	ITGLMX	+	Maximum number of unsteady-flow stabilization computations in each of several progressively increasing time steps, default value is 10.
6	IDTMX	+	Maximum number of iterations for ALPH=0 for the flow stabilization procedure, default value is 20.
7	IDTTMX	+	Maximum number of iterations for ALPH \neq 0 for the flow stabilization procedure, default value is 10.
8	ITADVL	+	Number of iterations for characteristic line, default value is 1.

Record AO

Required

<u>Field</u>	<u>Variable</u>	<u>Value</u>	<u>Description</u>
0	ID	AO	Record identification.
1	TEXL	+	Link number (name) on which the cross section to be on output is located. (To be used with Record AN.)
2	TEXL	+	Cross section member (name) for which data output is required. (To be used which Record AN.)

Record AT

Required

<u>Field</u>	<u>Variable</u>	<u>Value</u>	<u>Description</u>
0	ID	AT	Record identification.
1	TTBEG	+	Time of beginning of simulation, consistent with INDTI in record AB
2	TTEND	+	Time of end of simulation, consistent with INDTI in record AB
3	TDELTA	+	Time step, consistent with INDTI in record AB.

Record BE

Required

<u>Field</u>	<u>Variable</u>	<u>Value</u>	<u>Description</u>
0	ID	BE	Record identification.
1	EPSHYD	+	Threshold value of water surface changes for terminating global iterations in steady-flow computation, default value is 0.005 m
2	EPSQ	+	Threshold value of discharge changes for terminating global iterations in steady-flow computation, default value is 0.1 cms.
3	DRYQ	+	No use
4	EPSDYM	+	Threshold value of node water surface changes for terminating global iterations in unsteady-flow computation, default value is 0.001 m.
5	EPSYB	+	Threshold value of water surface level change corresponding to the specified time interval during the flow stabilization, default value is 0.01 m.
6	FDELTB	+	Base time interval for flow stabilization procedure.
7	A0	+	Value of the coefficient A0 for IDSBC=2.
8	B0	+	Value of the exponent B0 for IDSBC=2.

Record BL

Required

<u>Field</u>	<u>Variable</u>	<u>Value</u>	<u>Description</u>
0	ID	BL	Record identification.
1	LNKNAM	+	Integer "name" of link.
2	II	+	Number of computation points on link.
3	MU	+	Integer "name" of node at upstream end of link.
4	MD	+	Integer "name" of node at downstream end of link.
5	RATEP	+	

Record BP

Required

<u>Field</u>	<u>Variable</u>	<u>Value</u>	<u>Description</u>
0	ID	BP	Record identification.
1	SGRAV	+	Specific density of sediment, default value is 2.65.
2	POROS	+	Porosity, default value is 0.4.
3	THETA	+	Time weighting factor in de St. Venant equation.
4	PHI	+	Space weighting factor in de St. Venant equation.
5	S01	+	Initial value of bed slope.
6	DYINIT	+	Threshold value of node water surface changes for initial computation.

Record BS

Required

<u>Field</u>	<u>Variable</u>	<u>Value</u>	<u>Description</u>
0	ID	BS	Record identification.
1	LNAME	+	Integer “name” of link.
2	I	+	Number of computation point.
3	NNSEC	+	Number of section type associated with this point.
4	NNSD	+	Number of size fractions associated with this point.
5	ISWIT2	0	The current station is not a control section; therefore, no boundary condition is imposed there.
		1	The current station is a control section; therefore, the water surface elevation is a known function of the discharge (boundary condition) at this station.
6	ITYP2	0 or 1	If ISWIT2=0.
		1	If ISWIT2=1.

Record CF

Required for suspended sediment concentration computations

<u>Field</u>	<u>Variable</u>	<u>Value</u>	<u>Description</u>
0	ID	CF	Record identification.
1	CLNAME	+(-)	Link number (name), same as LNAME in Record BS. (Last one of Record CF should add a minus sign.)
2	CI	+	Cross section number, same as I value in Record BS. (In the order from downstream towards upstream.)
3	CONTA	+	Starting concentration value of suspended sediment. ($10^{-6} \text{ m}^3/\text{m}^3$)
4	FFX	+	Longitudinal dispersion coefficient (m^2/sec), superseded when IFXZ=1 in Record AI.
5	FFZ	+	Transverse dispersion coefficient (m^2/sec), superseded when IFXZ=1 in Record AI.

Record CT

Required for suspended sediment concentration computations

<u>Field</u>	<u>Variable</u>	<u>Value</u>	<u>Description</u>
0	ID	CT	Record identification
1-7	ITITLE		Define title of suspended sediment concentration computations. The text may include suspended sediment concentrations, corresponding boundary conditions, dispersion coefficients and starting concentration values, and will be echoed in the output file of NETS11.OUT.

Record GR

Required

<u>Field</u>	<u>Variable</u>	<u>Value</u>	<u>Description</u>
0	ID	GR	Record identification.
1,3,5,7,9	B0	+/-	Vertical coordinate (bottom elevation) of the data points that define the cross-sectional geometry at the current station.
2,4,6,8,10	CR	+/-	Lateral coordinate, measured from a reference point, of the data points that define the cross-sectional geometry at the current station.

Record LA

Required for lateral flows

<u>Field</u>	<u>Variable</u>	<u>Value</u>	<u>Description</u>
0	ID	LA	Record identification
1	INLAT	0,+	Total number of entering points of lateral flows. Locations of lateral flows are defined in Record LI. The maximum value of INLAT can not exceed total number of cross sections.

Record LT

Required for lateral flow

<u>Field</u>	<u>Variable</u>	<u>Value</u>	<u>Description</u>
0	ID	LT	Record identification.
1-7	ITITLE		ASCII text to be echoed in the NETSTARS output files.

Record NS

Required

<u>Field</u>	<u>Variable</u>	<u>Value</u>	<u>Description</u>
0	ID	NS	Record identification.
1	NUMSEC	+(-)	Value same as in NNSEC of Record BS. A minus sign should be added to this value for the last record.
2	PNTS	+	Total stations (data points) for the cross sections(170 is the default value)
3	BEDC	-,0,+	Increment in bed elevation. (0 is the default value)
4	WIDC	+	Enlargement coefficient for the width. (1.0 is the default value)
5	CCLF	-,0,+	Left bank station for scour limit. (-9999 is the default value)
6	CCRT	-,0,+	Right bank station for scour limit. (+9999 is the default value)
7	CCBT	-,0,+	Bed elevation for scour limit. (-9999 is the default value)
8	CCTP	-,0,+	Bed elevation for scour limit. (+9999 is the default value)
9	CLOSS	+	Energy loss coefficient at the cross section due to expansion, contraction or bend when IUNST=4 in Record AB.

Record RI

Required for later flows

<u>Field</u>	<u>Variable</u>	<u>Value</u>	<u>Description</u>
0	ID	RI	Record identification
1	TDAT1	-,0,+	Time data in the time series, same as TDAT in Record CQ. (Time unit same as INDTI in Record AB.)
2-10	RILI	+	Lateral flows at the points as described in Record LI. Field 2 value (the first data) corresponds to the first Record LI and the INLAT-th Record LI. When the number of data points exceeds 9, the subsequent Record RI should start the data point in Field 2. Units for lateral flows CMS (metric) or CFS (English). The corresponding sediment inflow(tons/day) is specified in the 2*INLAT-th data field. If the value of AC1 in Record LI is non-zero, the sediment inflow is computed using the relationship of sediment inflow values lateral inflow.

Record SF

Required for sediment computation

<u>Field</u>	<u>Variable</u>	<u>Value</u>	<u>Description</u>
0	ID	SF	Record identification.
1	F	+	Number of size fractions defined for the current study, $1 \leq F \leq 10$.

Record SN

Required for sediment routing

<u>Field</u>	<u>Variable</u>	<u>Value</u>	<u>Description</u>
0	ID	SN	Record identification
1	NODNUM	+(-)	Node number. (A minus sign should be added to this value for the last record)
2	NODSTT	+	Sediment size fraction number, value same as NSEDTT in Record SP.
3	PTYP	$0 \leq \text{PTYP} \leq 1$	Percent of sediment smaller than the corresponding size fraction per sieve analysis. If total number of size fractions exceeds 8, the subsequent Record SN should start the data in Field 3.

Record ST

Required

<u>Field</u>	<u>Variable</u>	<u>Value</u>	<u>Description</u>
0	ID	ST	Record identification
1-7	ITITLE		Define title of sediment routing computation. The text may include data, sources, sieve analysis, and sampling time of sediment data, limit for scour, total number of stream tables, etc, and will be echoed in the output file of NETS02.OUT.

Record TT

Required

<u>Field</u>	<u>Variable</u>	<u>Value</u>	<u>Description</u>
0	ID	TT	Record identification.
1-7	ITITLE		ASCII text to be echoed in the NETSTARS output files.

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附錄五

審查意見與處理情形

E.1 期中審查意見及辦理情形對照表

審查委員	意見	辦理情形
蔡教授長泰	<ol style="list-style-type: none"> 1. 水庫沉滓運移模式已完成水理模式部份，在進一步發展沈滓模式時請注意水庫入流泥砂以細粒沈滓為主，淤積也以細粒沈滓為主，其淤積現象宜為沈滓模式之重點。 2. 石門水庫除主流外，其兩側有多條支流匯入，其泥砂輸入對水庫局部淤積的影響也宜加入分析。 3. 石門水庫上游的榮華壩容量大，且尚未淤滿，在演算石門水庫沈滓運移之輸入條件(邊界條件)時，亦應列入考慮。 	<ol style="list-style-type: none"> 1. 遵照辦理。 2. 遵照辦理。 3. 遵照辦理。
林教授國峰	<ol style="list-style-type: none"> 1. 本計畫所發展之沈滓運移模式為何用二維正交曲線座標系統？請說明。 2. 曲線座標系統與卡氏座標系統之轉換方程式沒有推導提供於報告中？ 3. 曲線座標系統的使用符號恐有和卡氏座標系統混淆之處，請考慮更動。 4. 部分符號未定義，請補充。 5. 模式測試驗證用到潰壩案例，是否有必要，請斟酌。 	<ol style="list-style-type: none"> 1. 利用曲線座標系統或卡式座標系統均可處理目前的課題，本研究選用曲線座標的原因在於此系統處理不規則邊界問題較為容易。 2. 將於期末報告中補列。 3. 因為控制方程變數眾多，更動符號表示方式可能會使方程式表達趨於繁雜。目前在水庫沉滓運移模式之變數均建構於曲線座標系統，報告中對於每個變數均詳加定義說明，以免混淆。 4. 遵照辦理。 5. 誠如委員所言，潰壩與水庫淤砂無直接無關，故模式測試案例刪除潰壩的部分。
蔡副局長茂明	<ol style="list-style-type: none"> 1. 潰壩案例中請加入探討模擬榮華壩潰壩後，導致下游容量較小之義興壩一併潰壩之分析 	<ol style="list-style-type: none"> 1. 誠如林教授所言，潰壩與水庫淤砂無直接無關，本研究還是將研究重點著重於水庫淤砂之模擬，潰壩問題暫不考慮。

E.2 期末審查意見及辦理情形對照表

審查委員	意見	辦理情形
李理事長三畏	<ol style="list-style-type: none"> 1. 計畫預定工作與報告內容相符。 2. 如同保育事業組王工程師所言，建議於報告中增列一章，說明資料蒐集之相關內容。 3. 建議報告中，增列模式使用手冊之運用說明。 4. 請於報告中補列摘要。 5. 計畫名稱中採用”運移”兩字是否恰當。 	<ol style="list-style-type: none"> 1. 謝謝指教。 2. 經與王工程師溝通請教後，報告內容之編排方式已依委員意見辦理。 3. 已於報告中增列一個章節，說明及評估水庫沉滓運移模式、GSTARS 3.0 模式及 NETSTARS 模式，使模式使用者能更具體明瞭各模式之功能與應用範圍。 4. 遵照委員意見辦理。 5. 文獻中探討輸砂問題時，多用此描述。
賴博士進松	<ol style="list-style-type: none"> 1. 肯定謝組長推動此計畫，藉由評估美方發展的 GSTARS 3.0 模式，可增進中美雙方的互動。 2. GSTARS 3.0 版中，雖然加入岸壁沖淤、彎道模擬之功能，但是對於有較寬闊庫區之水流流況仍有其限制存在。雖然此模式已應用於大埔水庫及阿公店水庫之模擬，但也僅能展現”擬二維”的特性。就石門水庫而言的模擬而言，GSTARS 3.0 仍有其不足，故二維模式是有其發展的必要。 3. 對於水庫清淤操作課題而言，模擬石門水庫相關案例是有其必要性的，包括淤澱位置、營運管理等。但是用以檢定驗證模式中相關參數的現場量測資料或是水工試驗資料的取得，將會非常困難。未來如何推動與獲得數據，可在報告建議中提出。 	<ol style="list-style-type: none"> 1. 2. 謝謝指教。 3. 遵照委員意見辦理。研究思考可行之方案，並於未來報告中說明。

	大，歷年均在該處清淤，以水庫營運及砂石利用觀點，該處地點應為適當。但以有效清淤而言，該地點是否為最佳地點，或何處為最佳地點，請一併研究後，提供北水局最為以後清淤計畫參考。	5. 將此工作列為明年重點工作。
保育事業組	本計畫期末報告中，基本資料蒐集散在各章節中，建議將此部分整理為一個專章說明	經與保育事業組王工程師溝通請教後，報告內容之編排方式已依委員意見辦理。
綜合計畫組	<ol style="list-style-type: none"> 1. NETSTAR 英文使用手冊內容中之錯別字，請修正 2. NETSRAR 模式使用公制或英制的單位？ 	<ol style="list-style-type: none"> 1. 遵照委員意見辦理。 2. 兩者均可。