

國際港埠建設水力技術工程論文發表會  
(水力 - 港埠 '94) 出國報告

International Conference on Hydro-  
Technical Engineering  
for Port and Harbor Construction  
(Hydro-Port'94)



交通部運輸研究所

中華民國八十三年十二月

## 交通部運輸研究所出版品摘要表

出版品名稱 中文：國際港埠建設水力技術工程論文發表會(水力-港埠'94)出國報告 英文：International Conference on Hydro-Technical Engineering for Port and Harbor Construction			
國際標準書號(或叢刊號)		政府出版品統一編號 009106830649	
		運輸研究所出版品編號 83 - 67 - 049	
主辦單位：交通部運輸研究所運輸工程組 主管：侯和雄 計畫主持人：侯和雄 研究人員：侯和雄			研究期間 自八十三年十月 至八十三年十二月
關鍵詞：深水合成堤(Deepwater Composite Breakwaters)，波力(Wave Force)，上舉力(Up-lift Force)，沈箱(Caisson)，波壓分佈(Wave Pressure Distribution)，底板反力(Bottom Slab Reverse Force)，拋石堤承載率(Bearing Capacity of the Rubble Mound)，整平精度(Leveling Precision)，衝擊碎波力(Impact Breaking Wave Force)，液化(Liquefaction)，地震反應分析(Earthquake Reaction Analysis)			
摘要：日本為世界港埠技術研究馳名的國家，此次作者得以出席日本運輸省港灣技術研究所及海岸發展技術研究所共同主辦之「國際港埠建設水力技術工程論文發表會」發表兩篇論文，針對深水合成防波堤之設計因素與不規則波波力試驗值與理論公式值比較，修正了合田(Goda)不規則波波力公式，獲致日本港研所研究學者共鳴，為此行最大之收穫。 經由國際海岸港灣工程界學者專家對港埠建設遭遇之技術上困難問題而研究成功解決之方案，經與各國工程師、研究者之研討而得共識，則此等解決方案或創見乃成為港埠建設之先進技術。 藉由是項國際會議，作者得與各國學者專家交換研究心得並充分瞭解港埠技術之發展趨勢，獲益良多。			
出版日期	頁數	工本費	本出版品取得方式
83年12月	40	100元	凡屬機密性或限閱性出版品均不對外公開。一般性出版品，公益機關團體及學校可函洽本所免費贈閱；私人及私營機關團體可按工本費價購。
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備註：本研究之結論與建議不代表交通部之意見			

# 行政院國家科學委員會補助國內專家學人出席國際 學術會議報告

報告人 姓名	侯 和 雄	服務機關 及 職 稱	交通部運輸研究所 工程組簡任十一職等功四組長
時間 會議: 地點	83年10月19 日至10月21 日 日本	本會核定 補助文號	(83)台會合字第22992號函 補助案編號84--5-1-I-172-001-AI
會議名稱	(中文)國際港埠建設水力技術工程論文發表會(水力-港埠'94) (英文)International Conference on Hydro-Technical Engineering for Port and Harbor Construction (Hydro- Port'94)		
發表論文題目	(中文)1.深水合成防波堤不規則波壓之試驗值與理論公式值之比較 研究  2.深水防波堤設計因子之研究  (英文)1.Experimental Study and Theoretical Comparison of Irregular Wave Pressures on Deepwater Composite Breakwaters  2.Study of Deep Water Breakwater Design Factors		

**國際港埠建設水力技術工程論文發表會**

**(水力 - 港埠'94) 出國報告**

**International Conference on Hydro-**

**Technical Engineering**

**for Port and Harbor Construction**

**( Hydro-Port'94 )**

報告人：侯和雄

交通部運輸研究所運輸工程組組長

日本運輸省港灣技術研究所以及海岸發展技術研究所，主辦此項港埠建造之水力技術工程國際會議（簡稱HYDRO-PORT'94）。HYDRO-PORT'94主要在增廣並提高實際港埠建設技術，透過世界各國之學者專家、工程師們之發表論文即可獲致技術資訊及工程經驗。

國際港埠建設水力技術工程論文發表會，其主題為HYDRO-PORT'94為針對世界各國港埠建設相關之水力技術問題進行研討與交換意見，出席國家均為世界港埠、港岸工程素有盛名者，如美國、日本、中華民國、荷蘭、美國、丹麥、比利時、印度、西班牙、韓國、南非等卅餘國提出相關論文，精選論文總數之三分之一發表，計有八十篇來自廿餘國港埠先進國家之學者與專家共同參與，論文分為波浪結構設計分析、港埠建設波浪變形分析、合成防

波堤之設計波力、港池水質與海水水質之改善對策、港口附近海岸之穩定、港口或水道淤淺與防止淤塞之對策量測，許多港埠建設發展中遭遇之技術困難問題及對策均由研究人員及工程師成功解決之範例提出研討，使港埠建設技術更為先進與發展。

報告人在會中共計發表兩篇論文，且為我中華民國之唯一主要代表宣讀論文者，中共雖有報名參與該會議但並無宣讀論文，報告人係唯一代表中國人在大會廳演講者，另外張金機先生亦有一篇論文，但列為Poster Session，並無須在大會宣讀，報告人發表之論文如下：

1. 深水合成防波堤不規則波壓之試驗值與理論公式值之比較研究。  
(本文在大會中宣讀)

Experimental Study and Theoretical Comparsion of Irregular Wave

2. 深水防波堤設計因子之研究

Study of Deep Water Breakwater Design Factors.

### 會議中發表之主要內容

HYDRO-PORT'94提供世界各國港埠建設相關水力技術問題之論文發表俾研討交換新知，其各主題及子題如下：

1. 提供結構設計之波浪分析與資料整理
  - a. 海岸波浪之觀測網
  - b. 觀測波浪之分析方法

c. 波浪預測

2. 提供港埠建設之波浪變形分析

a. 提供港埠設施設計用之波浪變形分析

b. 海洋波浪在它們變形中之方向效應

c. 海洋波浪變形之物理與數值模式之實際應用

3. 合成防波堤之設計波力

a. 波壓公式

b. 設計中衝擊波力之處理

c. 防波堤安定性之實測資料分析

d. 新型防波堤之發展

4. 港池水質與海水水質之改善對策

a. 海水環流與交換

b. 流況與輸送底質環之模式

c. 海水水質之環境影響評估

d. 港內海水水質之改善

5. 港口附近之海灘穩定

a. 港口附近海灘變化

b. 海灘之穩定對策

c. 人工海灘

d. 向岸離岸之底質輸送與海灘剖面之穩定

6. 港口及水道漂沙輸送與淤積等之淺化量測

a. 漂沙輸送與淤積之現場觀測

b. 淺化對策現場量測之個案研究

### 會後觀感

此項國際港埠建設水力技術工程論文發表會議甚具工程實務與理論應用價值，尤其港埠在增進海洋運輸與貨物裝卸上扮演一項很重要之角色，對世界工業與經濟之發展貢獻良多，有些港埠則在島或岬狀地形背後之遮蔽區域發展而成與那些在開敞海岸暴露在大浪及過量漂沙威脅之情況，大異其趣。但是仍然有許多港須建造在不可避免之沖積海岸而且軟弱與不安定的土壤之上。

許多港埠建設與發展過程中所遭遇到的技術上之困難發生在世界各國的海岸上而承受者不同的海洋與地質條件，經由世界各國工程師與研究人員成功的解決諸項技術問題，於是此等解決方案即變成港埠建設之先進的技術。

港埠建設在開發中的國家常冀求已開發國家之相關技術之轉移與技術合作，然而，技術之應用在各個國家中因有個別自然條件與社經環境，因此僅能只限於某種港埠規劃，期望之港埠效用、設計觀念以及建設條件等不同之指導準則。

港埠建設技術之順利轉移則有賴於埋藏在個別技術及其應用限制中固有的束縛之詳加瞭解，因此有必要藉由此次論文發表會來交換

換意見與看法，尤其針對港埠設施之合理設計與安全評估，以及海洋構造物所促成之環境影響評估等方面，以獲得共識。

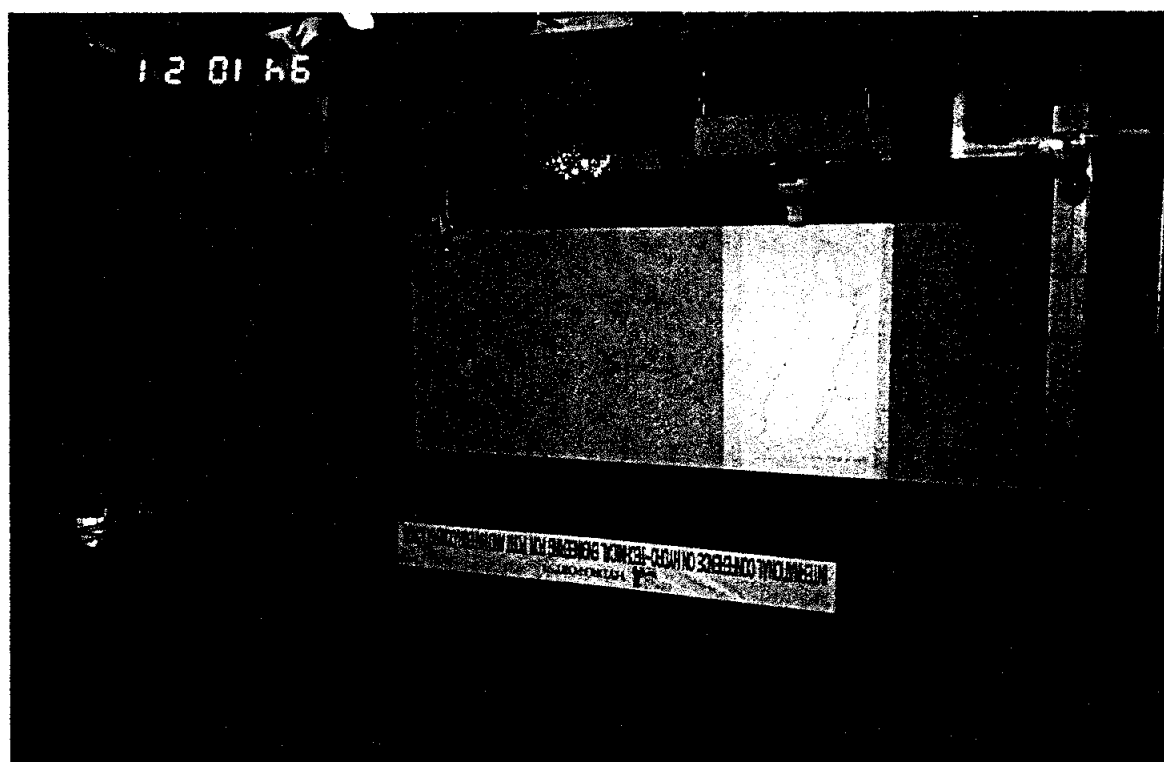
報告人發表之合成堤之波力分析與試驗研究，針對日本著名學者合田良實博士導演之公式加以試驗修正，使應用上更合理，且獲得與會學者之讚賞與共鳴，會後曾有日本運輸省水工部耐波研究室室長高橋博士、海洋開發研究所所長土田博士及日本東北大學名譽教授岩崎博士均與報告人深入探討修正公式之細節，此方面以試驗數據印證經驗設計公式在現代港埠設計理念上又跨出了一大步。

此次蒙國科會補助出席是項會議得與世界各國港埠海岸工程界名流交換研究心得與瞭解未來港埠技術之發展趨勢，頗有助益，殊甚感激，最後特對本所張所長家祝博士鼓勵同仁對外發表學術論文與研究作品，並推薦國科會申請補助出席國際會議之學術研究開闢心境表示由衷謝忱。

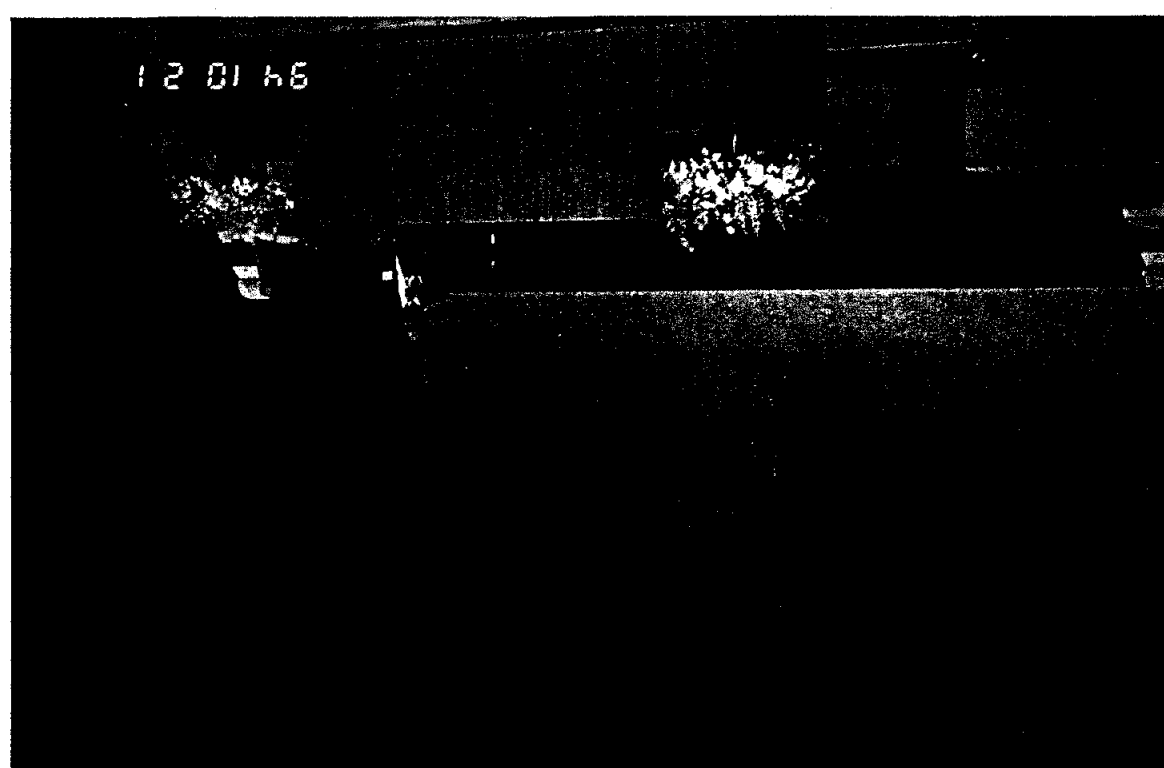
HYDRO-PORT'94之演講時程表列如附錄一，發表之兩篇論文列如附錄二及附錄三。



照片二 作者在大會中介紹台灣地區國際港深水化情況



照片一 作者在大會演講深水合成堤波力試驗研究





照片三 作者在研討室與丹麥教授Dr. Burcharth 研討  
" 深水防波堤設計因子之研究 "



照片四 作者在研討室與前日本港研所所長佐藤昭二博士與  
前水工部長田中則男博士研討深水堤防設計

# HYDRO-PORT'94

International Conference  
on  
Hydro-Technical Engineering  
for  
Port and Harbor Construction

## **CONFERENCE TIMETABLE (FINAL VERSION)**



OCTOBER 19-21, 1994  
YOKOSUKA, JAPAN

# CONFERENCE TIMETABLE

Wednesday, Oct.19

Opening Ceremony 10:00 - 10:30																	
Coffee Break 10:30 - 10:50																	
Keynote Lecture 10:50 - 11:50	A Plea for Engineering-minded Research Efforts in Harbor and Coastal Engineering Professor Yoshimi Goda, Yokohama National University, Japan																
Lunch 11:50 - 13:00																	
Technical Session 1 on Theme 1 13:00 - 15:25	<p>Theme : Acquisition and Analysis of Wave Information for Structural Design Chairman : Akira Watanabe, University of Tokyo, Japan Secretary : Toshihiko Nagai, Port and Harbour Research Institute, Japan</p> <table> <tr> <td>(1-1) 13:00-13:20</td><td>Introduction of Japanese NOWPHAS System and its Recent Topics (Nationwide Ocean Wave information network for Ports and Harbours) T.Nagai, K.Sugahara, N.Hashimoto, T.Asai, S.Higashiyama and K.Toda, Japan</td></tr> <tr> <td>(1-2) 13:20-13:40</td><td>Wave Monitoring System of the Korea Maritime and Port Administration K.S.Park, D.Y.Lee, C.S.Kim, S.W.Kang, K.S.Bahk, K.C.Jeon, S.I.Kim, J.S.Shim and B.C.Oh, Korea</td></tr> <tr> <td>(1-3) 13:40-14:00</td><td>Cooperation on the Improvement of the Wave Measurement and Analysis Technology between PHRI and KORDI D.Y.Lee<sup>1</sup>, K.S.Bahk<sup>1</sup>, K.S.Park<sup>1</sup>, B.C.Oh<sup>1</sup>, K.D.Seo<sup>1</sup>, T.Nagai<sup>2</sup>, N.Hashimoto<sup>2</sup>, T.Asai<sup>2</sup> and T.Horie<sup>2</sup>; <sup>1</sup>Korea, <sup>2</sup>Japan</td></tr> <tr> <td>(1-4) 14:00-14:20</td><td>Reliability of Wave Forecasting by Autoregressive Model Represented in Wave Energy H.Ohashi, S.Akamura, M.Suzuki and H.Inada, Japan</td></tr> <tr> <td>(1-5) 14:20-14:40</td><td>On the Extreme Wave Height Analysis H.F.Burcharth and Z.Liu, Denmark</td></tr> <tr> <td>(1-6) 14:40-15:00</td><td>Evaluation of Directional Wave Measurements - a Comparative Field Exercise A.van Tonder and J.Davies, South Africa</td></tr> <tr> <td>(1-7) 15:00-15:20</td><td>Measured Transformation of Deep Water Wave Spectra Across a Shallow Coral Reef Flat D.D.McGehee, U.S.A.</td></tr> <tr> <td>(Remarks) 15:20-15:25</td><td></td></tr> </table>	(1-1) 13:00-13:20	Introduction of Japanese NOWPHAS System and its Recent Topics (Nationwide Ocean Wave information network for Ports and Harbours) T.Nagai, K.Sugahara, N.Hashimoto, T.Asai, S.Higashiyama and K.Toda, Japan	(1-2) 13:20-13:40	Wave Monitoring System of the Korea Maritime and Port Administration K.S.Park, D.Y.Lee, C.S.Kim, S.W.Kang, K.S.Bahk, K.C.Jeon, S.I.Kim, J.S.Shim and B.C.Oh, Korea	(1-3) 13:40-14:00	Cooperation on the Improvement of the Wave Measurement and Analysis Technology between PHRI and KORDI D.Y.Lee <sup>1</sup> , K.S.Bahk <sup>1</sup> , K.S.Park <sup>1</sup> , B.C.Oh <sup>1</sup> , K.D.Seo <sup>1</sup> , T.Nagai <sup>2</sup> , N.Hashimoto <sup>2</sup> , T.Asai <sup>2</sup> and T.Horie <sup>2</sup> ; <sup>1</sup> Korea, <sup>2</sup> Japan	(1-4) 14:00-14:20	Reliability of Wave Forecasting by Autoregressive Model Represented in Wave Energy H.Ohashi, S.Akamura, M.Suzuki and H.Inada, Japan	(1-5) 14:20-14:40	On the Extreme Wave Height Analysis H.F.Burcharth and Z.Liu, Denmark	(1-6) 14:40-15:00	Evaluation of Directional Wave Measurements - a Comparative Field Exercise A.van Tonder and J.Davies, South Africa	(1-7) 15:00-15:20	Measured Transformation of Deep Water Wave Spectra Across a Shallow Coral Reef Flat D.D.McGehee, U.S.A.	(Remarks) 15:20-15:25	
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Coffee Break 15:25 - 15:45																	
Technical Session 2 on Theme 2 15:45 - 18:10	<p>Theme : Wave Transformation Analysis for Port Construction Chairman : Robert G.Dean, University of Florida, U.S.A. Secretary : Yasumasa Suzuki, Port and Harbour Research Institute, Japan</p> <table> <tr> <td>(2-1) 15:45-16:05</td><td>Field Application of Angular Spectrum Model to Directional Wave Transformation K.D.Suh, B.C.Oh and J.S.Shim, Korea</td></tr> <tr> <td>(2-2) 16:05-16:25</td><td>Time Domain Simulation of Directional Wave Propagation into Harbours O.G.Nwogu and E.P.D.Mansard, Canada</td></tr> <tr> <td>(2-3) 16:25-16:45</td><td>The Development and Application of a Computational Model of Directional Wave Transformation in Harbours N.P.Tozer and J.V.Smallman, U.K.</td></tr> <tr> <td>(2-4) 16:45-17:05</td><td>Applicability of Multi-directional Wave Experiment for Port Design Y.Suzuki, T.Hiraishi, T.Takayama and N.Ikeda, Japan</td></tr> <tr> <td>(2-5) 17:05-17:25</td><td>Field Applicability of Wave Models to Estimating the Wave Fields Outside and Inside a Harbor T.Shimizu, A.Ukai, Y.Kubo and M.Shimada, Japan</td></tr> <tr> <td>(2-6) 17:25-17:45</td><td>Development and Application of a Numerical Model for Wave Diffraction through Breakwater Gaps D.Anand and V.Sundar, India</td></tr> <tr> <td>(2-7) 17:45-18:05</td><td>Penetration of Long Waves into a Lagoon Harbour and Resulting Ship Motions V.Barthel, E.P.D.Mansard and D.D.MacDonald, Canada</td></tr> <tr> <td>(Remarks) 18:05-18:10</td><td></td></tr> </table>	(2-1) 15:45-16:05	Field Application of Angular Spectrum Model to Directional Wave Transformation K.D.Suh, B.C.Oh and J.S.Shim, Korea	(2-2) 16:05-16:25	Time Domain Simulation of Directional Wave Propagation into Harbours O.G.Nwogu and E.P.D.Mansard, Canada	(2-3) 16:25-16:45	The Development and Application of a Computational Model of Directional Wave Transformation in Harbours N.P.Tozer and J.V.Smallman, U.K.	(2-4) 16:45-17:05	Applicability of Multi-directional Wave Experiment for Port Design Y.Suzuki, T.Hiraishi, T.Takayama and N.Ikeda, Japan	(2-5) 17:05-17:25	Field Applicability of Wave Models to Estimating the Wave Fields Outside and Inside a Harbor T.Shimizu, A.Ukai, Y.Kubo and M.Shimada, Japan	(2-6) 17:25-17:45	Development and Application of a Numerical Model for Wave Diffraction through Breakwater Gaps D.Anand and V.Sundar, India	(2-7) 17:45-18:05	Penetration of Long Waves into a Lagoon Harbour and Resulting Ship Motions V.Barthel, E.P.D.Mansard and D.D.MacDonald, Canada	(Remarks) 18:05-18:10	
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(2-2) 16:05-16:25	Time Domain Simulation of Directional Wave Propagation into Harbours O.G.Nwogu and E.P.D.Mansard, Canada																
(2-3) 16:25-16:45	The Development and Application of a Computational Model of Directional Wave Transformation in Harbours N.P.Tozer and J.V.Smallman, U.K.																
(2-4) 16:45-17:05	Applicability of Multi-directional Wave Experiment for Port Design Y.Suzuki, T.Hiraishi, T.Takayama and N.Ikeda, Japan																
(2-5) 17:05-17:25	Field Applicability of Wave Models to Estimating the Wave Fields Outside and Inside a Harbor T.Shimizu, A.Ukai, Y.Kubo and M.Shimada, Japan																
(2-6) 17:25-17:45	Development and Application of a Numerical Model for Wave Diffraction through Breakwater Gaps D.Anand and V.Sundar, India																
(2-7) 17:45-18:05	Penetration of Long Waves into a Lagoon Harbour and Resulting Ship Motions V.Barthel, E.P.D.Mansard and D.D.MacDonald, Canada																
(Remarks) 18:05-18:10																	
Reception 18:20 - 20:00	On the 5th Floor of the Yokosuka Prince Hotel																

Thursday, Oct.20

Technical Session 4 on Theme 4 9:00 - 11:25	Theme : Harbor Water Quality and Seawater Quality Improvement Chairman : Desmond A. Lord, D.A. Lord and Associates, Australia Secretary : Kazuo Murakami, Port and Harbour Research Institute, Japan	
	(4-1) 9:00- 9:20	Characteristics of Wind Induced Upwelling in Tokyo Bay based on Analysis of NOAA-AVHRR data S. Ueno, K. Nadaoka, H. Ohtani and H. Katsui, Japan
	(4-2) 9:20- 9:40	3-Dimensional Modelling of Heated Water Discharges into Coastal Waters of Yellow Sea S. W. Kang and T. S. Jung, Korea
	(4-3) 9:40-10:00	Development of Water-Intake Works with Submerged Mound (WWSM) A. Nakayama, M. Yamamoto, J. Yamamoto and A. Moriguchi, Japan
	(4-4) 10:00-10:20	Water Quality in Penang Harbor P. M. Sivalingam and S. V. Charles, Malaysia
	(4-5) 10:20-10:40	Environmental Monitoring of the Lagunar Complex of the South Region of the Santa Catarina State D. Accetta, W. S. S. Dias, B. M. Vargas and J. A. dos. Santos, Brazil
	(4-6) 10:40-11:00	Channel Experiments on Coastal Water Purification by Stone Bed with Biofilm Y. Hosokawa, T. Ootsuki and C. Niwa, Japan
	(4-7) 11:00-11:20	Sand Covering for Improving Quality of Bottom Sediments in Japanese Harbors S. Inoue, T. Horie, K. Murakami, Y. Hosokawa, S. Sato and Y. Segi, Japan
	(Remarks) 11:20-11:25	
Coffee Break 11:25 - 11:40		
Special Lecture 1 11:40 - 12:20	Management of Coastal Waters in Western Australia : The Use of Integrated Models Dr. Desmond A. Lord, Director of D.A. Lord and Associates, Australia	
Lunch 12:20 - 13:30		
Technical Session 5 on Theme 5 13:30 - 15:55	Theme : Beach Stabilization in the Vicinity of a Harbor Chairman : Norio Tanaka, Nippon Tetrapod Co., Ltd., Japan Secretary : Kazumasa Katoh, Port and Harbour Research Institute, Japan	
	(5-1) 13:30-13:50	Change of the Ag. Nikolaos Beach in the Vicinity of a New Harbor for Small Crafts C. I. Moutzouris, Greece
	(5-2) 13:50-14:10	Protection against Shore Erosion and Channel Shoaling at Port Madero, MEXICO J. M. Montoya R. <sup>1</sup> , J. R. Vera S. <sup>1</sup> and S. Sato <sup>2</sup> ; <sup>1</sup> Mexico, <sup>2</sup> Japan
	(5-3) 14:10-14:30	Control of Littoral Drift in Caldera Port, Costa Rica J. G. Rodoriguez P. <sup>1</sup> and K. Katoh <sup>2</sup> ; <sup>1</sup> Costa Rica, <sup>2</sup> Japan
	(5-4) 14:30-14:50	Physical Impact of Bilbao Harbour New Breakwater on Adjacent Beaches J. P. Sierra, J. A. Jiménez, A. Sánchez-Arcilla, J. M. Picó, A. Viñuales and E. J. Villanueva, Spain
	(5-5) 14:50-15:10	Littoral Drift in Fishing Ports and Approach Channels: Problems and Countermeasures M. Fukuya, N. Takaki, K. Ota, S. Harikai and M. Ikeda, Japan
	(5-6) 15:10-15:30	Stabilization of Beach in Integrated Shore Protection System K. Katoh, S. Yanagishima, S. Nakamura and M. Fukuta, Japan
	(5-7) 15:30-15:50	Scour around the Head of a Vertical-Wall Breakwater T. Gökçe, B. M. Sumer and J. Fredsøe, Denmark
	(Remarks) 15:50-15:55	
Poster Session 16:00 - 18:00	Council Rooms of Yokosuka Industrial Plaza on the 3rd Floor of the Bay Square Yokosuka Building (Posters will be exhibited from 9:00 to 18:00.)	

Friday, Oct.21

<p>Technical Session 3 on Theme 3 9:00 - 12:20</p> <p>→</p>	<p>Theme : Design Wave Forces on Composite Breakwaters Chairman : Hans F. Burcharth, University of Aalborg, Denmark Secretary : Shigeo Takahashi, Port and Harbour Research Institute, Japan</p>	
	(3-1) 9:00- 9:20	Caisson Breakwaters: Integrated Design and Wave Load Specifications H. Oumeraci, A. Kortenhaus and P. Klammer, Germany
	(3-2) 9:20- 9:40	Experimental Study and Theoretical Comparison of Irregular Wave Pressures on Deepwater Composite Breakwaters H. S. Hou, Y. D. Chiou and C. H. Chien, Chinese Taipei
	(3-3) 9:40-10:00	A Proposal of Impulsive Pressure Coefficient for the Design of Composite Breakwaters S. Takahashi, K. Tanimoto and K. Shimosako, Japan
	(3-4) 10:00-10:20	A Comparative Study on Wave Forces and Overtopping of Caisson Breakwaters J. Juhl, Denmark
	(3-5) 10:20-10:40	Design of Harbour Entrances: Breakwater Design and Vessel Safety M. W. McBride, J. V. Smallman and N. W. H. Allsop, U.K.
	(Remarks) 10:40-10:45	
	(Coffee Break) 10:45-10:55	
	<p>Chairman : Katsutoshi Tanimoto, Saitama University, Japan Secretary : Shigeo Takahashi, PHRI, Japan</p>	
	(3-6) 10:55-11:15	A New Type of Breakwater with a Step-Shaped Slit Wall R. Fujiwara, T. Yoshida, K. Kurata, S. Kakuno and K. Oda, Japan
	(3-7) 11:15-11:35	Hydraulic Experiments for Basic Design of Offshore Breakwater for Protecting Artificial Island D. S. Lee, W. S. Park, K. D. Suh and Y. M. Oh, Korea
	(3-8) 11:35-11:55	New Types of Breakwater Two Projects in MONACO R. Bouchet, P. Cellario and J. L. Isnard, Monaco
	(3-9) 11:55-12:15	Field Demonstration Test on a Semi-Circular Breakwater H. Sasajima, T. Koizuka, H. Sasayama, Y. Niidome and T. Fujimoto, Japan
	(Remarks) 12:15-12:20	
<p>Lunch 12:20 - 13:30</p>		
<p>Special Lecture 2 13:30 - 14:10</p>		<p>Engineering Devices for a Smooth Port Operation with Focus on Developing Countries Professor Isao Irie, Kyushu University, Japan</p>
<p>Coffee Break 14:10 - 14:30</p>		
<p>Technical Session 6 on Theme 6 14:30 - 16:15</p>	<p>Theme : Countermeasures against Shoaling due to Siltation and Sedimentation in the Harbor and Waterway Chairman : Isao Irie, Kyushu University, Japan Secretary : Hiroichi Tsuruya, Port and Harbour Research Institute, Japan</p>	
	(6-1) 14:30-14:50	Criteria and Methods to Determine Navigable Depth in Hyperconcentrated Sediment Layers W. R. Parker and P. M. Hooper, U.K.
	(6-2) 14:50-15:10	Field Surveys on Siltation-Prevention Effects in Waterway and Anchorage by Submerged Walls T. Kihara, H. Sasajima, K. Yoshinaga, T. Koizuka, H. Sasayama, H. Yoshinaga and T. Fujimoto, Japan
	(6-3) 15:10-15:30	Case Study on Channel Improvement in the Mouth of the Yongjiang River J. Jiang, China
	(6-4) 15:30-15:50	Field Survey on Estuarine Mud Transport Process around Navigation Channel, Banjarmasin, Indonesia H. Tsuruya, K. Murakami, K. Nagai and I. Irie, Japan
	(6-5) 15:50-16:10	Modelling Cohesive Sediment Transport in Tidal Waters K. P. P. Pathirana, J. C. S. Yu and J. Berlamont, Belgium
	(Remarks) 16:10-16:15	
<p>Closing Ceremony 16:20 - 16:40</p>		

# Experimental Study and Theoretical Comparison of Irregular Wave Pressures on Deepwater Composite Breakwaters

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## ABSTRACT

1:49 model tests are carried out to investigate the irregular wave pressures and up-lift on deep-water composite breakwaters, of which prototypes are separately located at water depth -32.6 m, -28.6 m. Using ten sets of pressure gauges and with sampling rate  $2000H_z$ , wave pressures and up-lift on upright section of composite breakwaters are sampled.

Utilizing zero-up crossing method, we analyze the statistical values of wave pressures to demonstrate the relationship between  $P_{max}/P_{1/3}$ ,  $P_{1/10}/P_{1/3}$ ,  $P_{rms}/P_{1/3}$ ,  $P_{mean}/P_{1/3}$  and  $H_o/d$ . Through Kolmogorov-Smirnov goodness-of-fit test with experimental data, it reveals that Weibull is a more suitable function to describe the probability distribution of wave pressures than Log-normal and Rayleigh function. The comparison between experimental data and three theoretical formulas are made; it shows 3rd order standing wave pressure formula is more suitable to describe the distribution of experimental data for non-overtopping cases. A modification is suggested in this study for the purpose of application to Goda's formula.

**Key Words:** Wave Pressure, Up-lift, Deepwater Composite Breakwater

## LIST OF SYMBOLS

d	water depth in front of breakwater (m)
h	distance from the design water level to the bottom of upright section
F	wave force (Kg/m)
$F_{max}$	maximum wave force (Kg/m)
$h_c$	crest elevation above still water (m)

$H_o$	deep water wave height (m)
$L_o$	deep water wave length (m)
$P$	wave pressure ( $g/cm^2$ )
$P_{max}$	maximum wave pressure ( $g/cm^2$ )
$P_{1/10}$	average value of highest one-tenth wave pressures
$P_{1/3}$	significant wave pressure ( $g/cm^2$ )
$P_{rms}$	root-mean-square value of wave pressures
$r$	specific weight of water( $Kg/m^2/sec^2$ )
$T$	wave period (sec)
$F/rd^2$	dimensionless wave force parameter
$P/rd^2$	dimensionless wave pressure parameter
$H_o/d$	dimensionless wave height
$H_o/L_o$	wave steepness
$d/L_o$	relative water depth

## 1. INTRODUCTION

Since the tendency of sea-transport has developed so prosperously in our country as well as in the world, it is necessary to have a deepwater harbor for loading bulk cargo and industrial materials. Besides, the construction of deep water harbor can provide a reclaimed land for the use of industrial park and airport. In Taiwan, we have many experiences for designing the composite breakwater about general water depth; but, those experiences in deep water region are still immature. Furthermore, a breakwater's failure or not and its cost highly depends on a precisely design especially in the deepwater region.

As we know, wave force acting on the upright section of composite breakwater is one of main external factors for the design. The standing waves almost developed in the front of deepwater composite breakwater. Many theoretical analyses of standing wave pressure on the vertical wall have been made by the researchers. For regular standing waves, the acquainted theoretical analyses include the Sainflou's formula<sup>[1]</sup>, small amplitude standing wave pressure formula<sup>[1]</sup> and Tadjbakhsh-keller(1960)<sup>[2]</sup> 3rd order standing wave pressure formula. For irregular wave, Goda(1974)<sup>[3]</sup> suggests an empirical wave pressure formula for the composite breakwater. What extent the confidence would be, when the above formulas are utilized to calculate the wave pressures on the composite breakwater acting in the deeper water. Therefore, this study tries to investigate the wave pressure and up-lift on the upright section through the irregular wave model tests. Then, the characteristics of deepwater wave pressures and the feasibilities of above wave pressure formulas to the deepwater breakwater discussed.

## 2. EXPERIMENT

1:49 model tests of irregular wave are carried out in the laboratory of Department of Hydraulics and Ocean Engineering, National Cheng Kung University, Taiwan, R.O.C.. The prototypes of composite breakwater's cross-sections as shown in Fig-1 & Fig-2 respectively, are offered by the Transportation Engineering Division Institute of Transportation. According to Froude law the dimensions of model scales are obtained as Table-1.



Table-1 Dimensions of model scales

Item	Prototype			Model		
Scale	1			1/49		
Designed wave height $H_{1/3}$	8 m			16.33 cm		
Period $T_{1/3}$ (sec)	9	11	13	1.29	1.57	1.86
Water depth d	32.6 m			66.5 cm		
	28.6 m			58.4 cm		
Caisson weight	1363.13 ton/m			567.73 Kg/m		
Wave pressure	0.49 ton/m <sup>2</sup>			1 g/cm <sup>2</sup>		

Layout of experiment is shown in Fig-3. The shell of model caisson is made of iron, and filled with sand to match the design weight as shown in Table-1. Ten sets of calibrated pressure gauges are mounted on caisson (6 sets on front face, 4 sets under the bottom) as shown in Fig-4. In the form of JONSWAP spectrum, irregular waves are generated with the above periods, in which the wave conditions are as follows:  $d/L_0=0.1082\sim 0.2563$  and  $H_0/d=0.08\sim 0.4$ . Meanwhile, the wave pressure data are measured at the rate of 2000Hz per channel and store for later analysis.

In addition, the crest elevation of parapet of caisson is lowered by 1 meter and 3 meters to investigate its influence on wave pressures and the up-lift. The detail classifications of tests are shown as Table-2.

Table-2 Classification of test cases

CASE	d		$d/L_0$	$h_c$		$h_c/d$
	Proto.(m)	Model(cm)		Proto.(m)	Model(m)	
A-1	32.6	66.5	0.2563, 0.1730, 0.1233	+ 10	+ 15.10	0.2270
A-2	32.6	66.5		+ 9	+ 13.06	0.1963
A-3	32.6	66.5		+ 7	+ 8.98	0.1350
B-1	28.6	58.4	0.2248, 0.1518, 0.1082	+ 10	+ 15.10	0.2587
B-2	28.6	58.4		+ 9	+ 13.06	0.2238
B-3	28.6	58.4		+ 7	+ 8.98	0.1538

### 3. EXPERIMENTAL RESULTS

#### (1) Dimensionless wave pressure

Fig-5 shows the typical results of the relationship between wave pressure and  $H_0/d$ . Paper selects  $P_2$  (just beneath the S.W.L.) and  $P_8$  (near the toe of caisson) to demonstrate above relation.

In general, both  $P_{1/3}/d$  of  $P_2$  and  $P_8$  present the increasing tendency with  $H_0/d$ . Nevertheless, for  $P_2$ , the increasing slope of  $P_{1/3}$  in the range of  $H_0/d < 0.2$  is steeper than that in the range of  $H_0/d > 0.2$ . This is considered as the effect of overtopping. As  $H_0/d > 0.22$ , the water surface of larger standing wave would higher than the crest of parapet, and the apparent overtopping would happen for larger wave in the wave train. This causes the instant reduction of  $P_{1/3}$  near the S.W.L. On the other hand, this disturbance induced by the overtopping of larger standing wave would be damped through the water depth. As a result, the increasing tendency of  $P_8$  seems not effected by the disturbance near the water surface. Since the minimum freeboard of the caisson is larger than the 0.4 times of maximum incident wave height, lowering the crest elevation by 1 meter and 3 meters made less effect on pressures under the same wave condition.

## (2) Statistical value of wave pressures

Utilizing zero-up crossing method, we analyze the statistical values of wave pressures and the results are noted as " $P_{\max}$ "、" $P_{1/10}$ "、" $P_{1/3}$ "、" $P_{\text{rms}}$ "、" $P_{\text{mean}}$ " respectively. For simplicity, paper selects gauge  $P_2$ 、 $P_8$  of CASE A-1、A-3、B-1、B-3 to demonstrate the relationship between  $P_{\max}/P_{1/3}$ 、 $P_{1/10}/P_{1/3}$ 、 $P_{\text{rms}}/P_{1/3}$ 、 $P_{\text{mean}}/P_{1/3}$  and  $H_0/d$ . The typical results are shown in Fig-6.

In Fig-6(A)(E),  $P_{\max}/P_{1/3}$  values show much more scattered than the others. The scattered ranges of  $P_{\max}/P_{1/3}$  are increasing with the increases of  $H_0/d$ . For gauge  $P_2$ ,  $P_{\max}/P_{1/3}$  varies from 1.2 to 1.8 and 1.2 to 2.3 for gauge  $P_8$ . The reason  $P_8$ 's range is larger than  $P_2$ 's will demonstrate in section(5).  $P_{1/10}/P_{1/3}$  shows the similar consequences as stated above. For gauge  $P_2$ ,  $P_{1/10}/P_{1/3}$  varies from 1.2 to 1.28 and 1.2 to 1.75 for gauge  $P_8$ . On the other hand,  $P_{\text{rms}}/P_{1/3}$  and  $P_{\text{mean}}/P_{1/3}$  values seem not effected by  $H_0/d$ , while  $H_0/d$  is increasing, their values remained constant.  $P_{\text{rms}}/P_{1/3}$  and  $P_{\text{mean}}/P_{1/3}$  values are equal to 0.75 and 0.7 respectively.

Through the investigation of wave pressure formulas, such as Sainflou's formula, much formulas describe that wave pressures are linearly proportional to wave height. Assuming the wave height distribution satisfies Rayleigh distribution; we can infer the statistical values of wave height, such as  $H_{\max}$ ,  $H_{1/3}$ , etc.. Base upon the assumptions stated above, the theoretical values of  $P_{\max}/P_{1/3}$ 、 $P_{1/10}/P_{1/3}$ 、 $P_{\text{rms}}/P_{1/3}$ 、 $P_{\text{mean}}/P_{1/3}$  are obtained and their values are equal to 1.61、1.27、0.7、0.625. Comparing to the experimental data, it shows theoretical ratio values are less than the experimental data, especially for  $P_{\max}/P_{1/3}$ . These results show that the Rayleigh distribution seems not a suitable function to describe the probability distribution of wave pressures. It inquires further investigation. The discussions are as followed.

## (3) The probability distribution of wave pressures

In order to calibrate the characteristics of wave pressure's probability distribution, paper selects gauge  $P_3$  to analyze its probability distribution; meanwhile, the results are comparing to Weibull、Log-normal and Rayleigh function.

Fig-7 shows the comparison between  $P_3$ 's exceedance probability distribution and the three theoretical functions. The results tell, when  $H_0/d$  is smaller, the distributions of three functions are in accordance with  $P_3$ 's, when  $H_0/d$  is larger, Weibull shows much more agree with  $P_3$ 's than the other two functions. In general, Rayleigh distribution gives some underestimate to small wave pressure when its probability is larger, on the other hand, it will overestimate large wave pressure when it's probability in smaller. The Log-normal distribution has the phenomenon just contrast to Rayleigh.

Through Kolmogorov-Smirnov goodness-of-fit test<sup>[4]</sup>, authors try to investigate the discrepancy between  $P_3$ 's distribution and the three functions. The results are recorded in Table-3. In Table-3, "n" value means the total number of wave pressures in a wave train which are combined together to evaluate the  $P_3$ 's probability distribution, "Dn" means the absolute maximum value which is evaluated by subtracting  $P_3$ 's cumulative probability distribution from theoretical function, "Dnx" is defined by Kolmogorov-Smirnov goodness-of-fit test theorem. The theorem defines, when Dn is smaller than Dnx; the test function is a suitable function to describing the distribution of test data. Through comparison of three functions' Dn values, it reveals that Weibull is a more suitable function to describe the probability distribution of wave pressures than Log-normal and Rayleigh function.

The results stated above are much the same with the results of Tang, etc. (1990)<sup>[5]</sup>, though their experiments are carried out in the regular depth water region. In authors' opinion, if we can calibrate the relationship between Weibull function and wave parameters, such as  $H_0/d$ ,  $d/L_0$ , etc., it may give a more realistic method to evaluate the irregular wave pressures.

Table-3 The Kolmogorov-Smirnov goodness-of-fit test

CASE	$d/L_0$	$H_0/d$	n	Weibull Dn	Log-normal Dn	Rayleigh Dn	K-S Dnx
A-1	0.2563	0.2861	84	0.0489	0.1678	0.1573	0.1167
		0.1194	98	0.0713	0.1704	0.0994	0.1082
A-3	0.2563	0.2935	86	0.0736	0.1774	0.2382	0.1154
		0.0826	88	0.0493	0.1237	0.1069	0.1141
B-1	0.1082	0.3657	97	0.0585	0.1715	0.1899	0.1086
		0.1061	100	0.0467	0.1014	0.2127	0.1070
B-3	0.1082	0.3442	98	0.0522	0.1511	0.2088	0.1081
		0.1037	100	0.0724	0.1370	0.2247	0.1070

#### (4) Pressures distribution on the front face of caisson

Pressure distributions on the front face of caisson are plotted in Fig-8. Taking significant wave height  $H_{1/3}$  as a regular wave height, the theoretical wave pressure distributions are plotted according to Sainflou's formula<sup>[1]</sup> (noted as "Sainflou"), small amplitude standing wave pressure formula<sup>[1]</sup> (noted as "1st"), and Tadjbakhsh-keller 3rd order standing wave pressure formula<sup>[2]</sup> (noted as "3rd").

The comparison shows; there is only a small difference between them when  $H_0/d$  is small. But when  $H_0/d$  gets larger, the discrepancy is very remarkable. Beneath still water level, Sainflou's formula gives larger pressure values than the other two, while 1st order approach is much closer to Sainflou's than 3rd order.

Conclusively, 3rd order approach is more suitable to describe the distribution of experimental pressure data, and the other two formulas would overestimate when  $H_0/d$  is larger.

#### (5) Maximum wave force and Up-lift force

The above mentioned formulas are derived for the regular waves. Thus, they shouldn't be utilized to calculate the maximum wave force induced by the irregular waves. Usually, Goda's empirical formula is used to compute the maximum wave force induced by irregular waves. So, the authors are to examine the applicability of Goda's formula to the calculation of maximum force on the deepwater composite breakwater.

The wave force on the front wall of caisson is evaluated by integrating the wave pressure from gauge  $P_1$  to  $P_7$  at the same time. After integration, a discrete time series for wave force is obtained as  $2000 H_z$ . Utilization of zero-up crossing method, a train of individual wave forces are thus analyzed and the maximum wave forces may be obtained as the maximum in that train of wave forces. The up-lift force is calculated from the pressure data of gauge  $P_7$  to  $P_{10}$  in the similar method.

From Fig-9 to Fig-10, the comparison of maximum wave force and up-lift force between the experimental data and those calculated from Goda's formula are separately presented in the sub-figures (A) and (B).

As shown in sub-figures (A), Goda's formula gives some overestimate to maximum force on the vertical wall while  $H_o/d$  increases. The discrepancy to overestimate also increases with  $H_o/d$ . Similarly, these discrepancies are due to the apparent overtopping of large waves as stated above. In other hand, up-lift forces in most of test runs seem to be underestimated by Goda's formula. It is due to the piling of water level behind the caisson induced by the overtopping of large waves. This water piling effect is not included in the Goda's formula. Thus, the up-lift force would be underestimated.

Base on the concerning experiments in this study, a modifying regression is made to the force ratio of experimental data ( $F_{lab}$ ) to Goda's formula ( $F_{Goda}$ ). Since the lowering of the caisson crest made less effect on the wave force, all experimental data of front wall force and up-lift force are separately plotted with  $H_o/d$  in Fig-11(A) and (B). Under the assumption of linear function of  $H_o/d$ , the regressing results are tabulated as Table-4. In the regression,  $R_{to}$  and  $\sigma$  separately represent the force ratio,  $F_{Lab}/F_{Goda}$ , and the standard deviation of force ratio.

Table-4 Modification formulas

$d/L_o = 0.1082 \sim 0.2563$	
Wave Force	
Upper limit	$R_{to} = -1.38(H_o/d) + 1.40$
Regression	$R_{to} = -1.38(H_o/d) + 1.16$
Lower limit	$R_{to} = -1.38(H_o/d) + 0.92$
Up-lift Force	
Upper limit	$R_{to} = 1.14(H_o/d) + 3.72$
Regression	$R_{to} = 1.14(H_o/d) + 1.08$
Lower limit	$R_{to} = 1.14(H_o/d) + 0.22$

To examine the feasibility of regression, the experimental data of CASE A-1( $T=9$  sec) and CASE B-1( $T=13$  sec) are used to compare with the modified data. The results are shown in Fig-12 and Fig-13. As Fig-12(A) shows, the experimental data located within the region of upper and lower limit line. The agreement between the modified data and experimental data is more satisfied than that shown in Fig-9 (A). As Fig-12(B) shows, modified data have a little overestimate. Nevertheless, the data are still within the region of upper and lower limit lines. The results show in Fig-13(A)(B) are similar to above. Therefore, the modification is reasonable for concerning cases in this study.

#### 4. CONCLUSIONS

For the prototype composite breakwater located at depth of 28.6 m and 32.6 m, the irregular wave force and up-lift on caisson are investigated by the model test. The wave condition are:  $d/L_0=0.1082\sim0.2563$ ,  $H/3=0.08\sim0.4$ . The results are summarized as followed:

- 1) Through Kolmogorov-Smirnov goodness-of-fit test with experimental data, it reveals that Weibull is a more suitable function to describe the probability distribution of wave pressures than Log-normal and Rayleigh function.
- 2) As treating the regular wave, the distribution of significant wave pressures  $P_{1/3}$  on front wall may be described by Tadjbakhsh-Keller 3rd order standing wave approximation.
- 3) While the apparent overtopping happened, the induced disturbance made the reduction of wave pressure near the S.W.L. and caused the wave force overestimation of Goda's formula.
- 4) The apparent overtopping might cause the pilling of water level behind the breakwater and, as a result, cause the underestimation of Goda's formula to the up-lift force.
- 5) For the experimental cases in this study, the modifying regression of force ratio to Goda's formula may obtain a reasonable agreement with the experimental data.

#### REFERENCES

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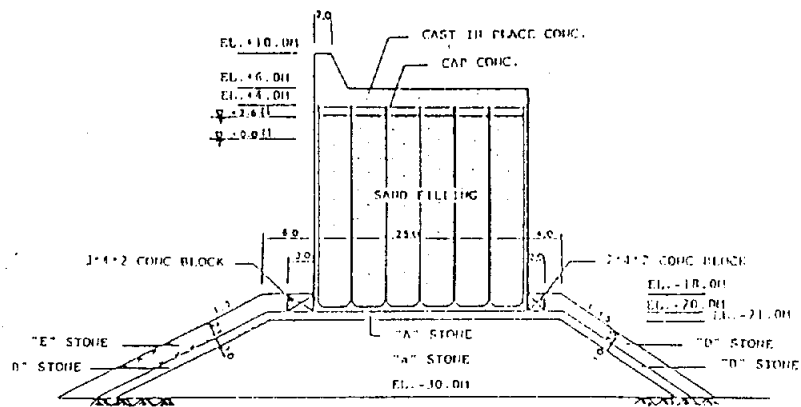


Fig-1 The design cross-section of composite breakwater

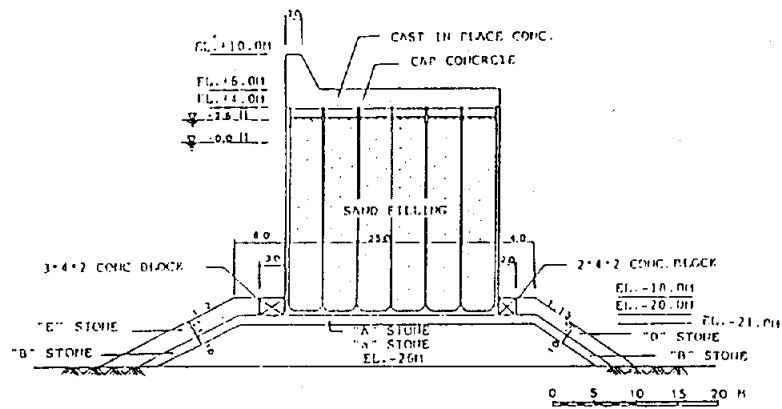


Fig-2 The design cross-section of composite breakwater

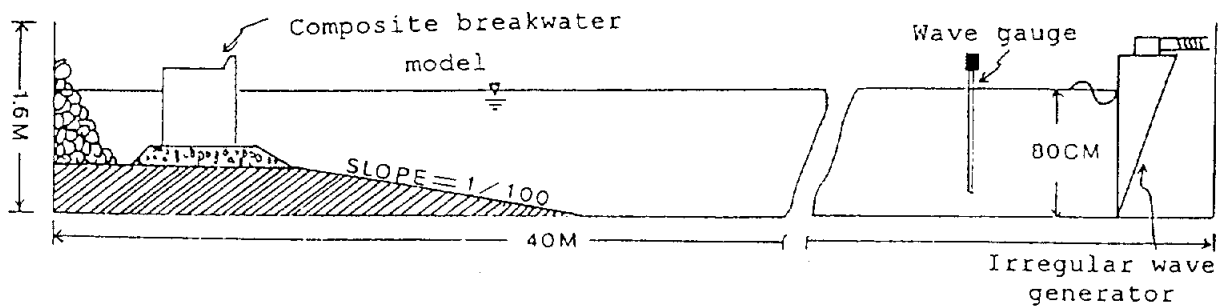


Fig -3 Layout of experiment

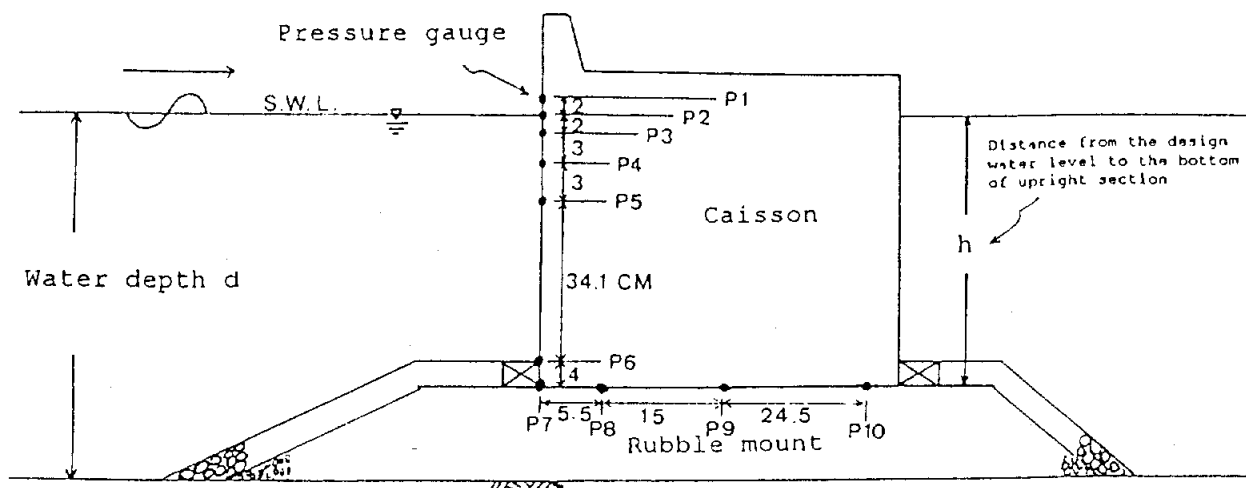


Fig-4 Layout of pressure gauges

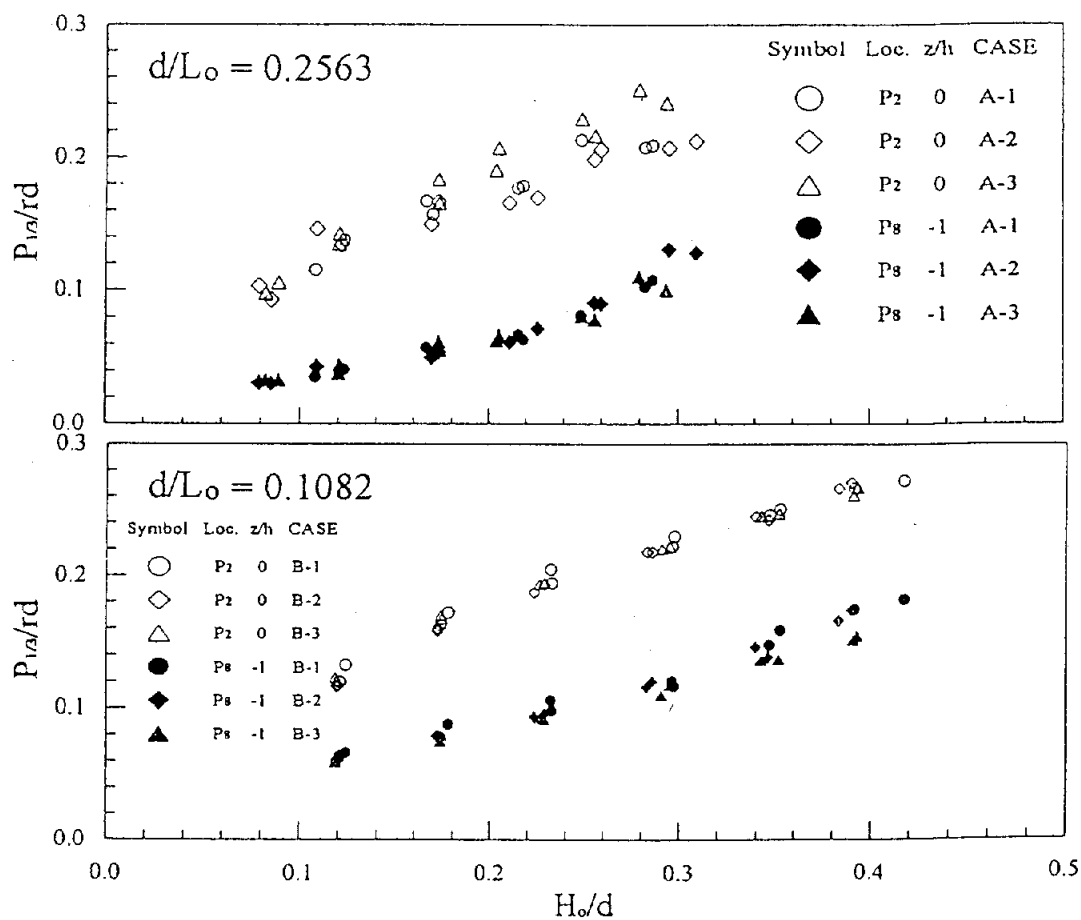


Fig-5 The relationship between  $P_{10}/rd$  and  $H_0/d$

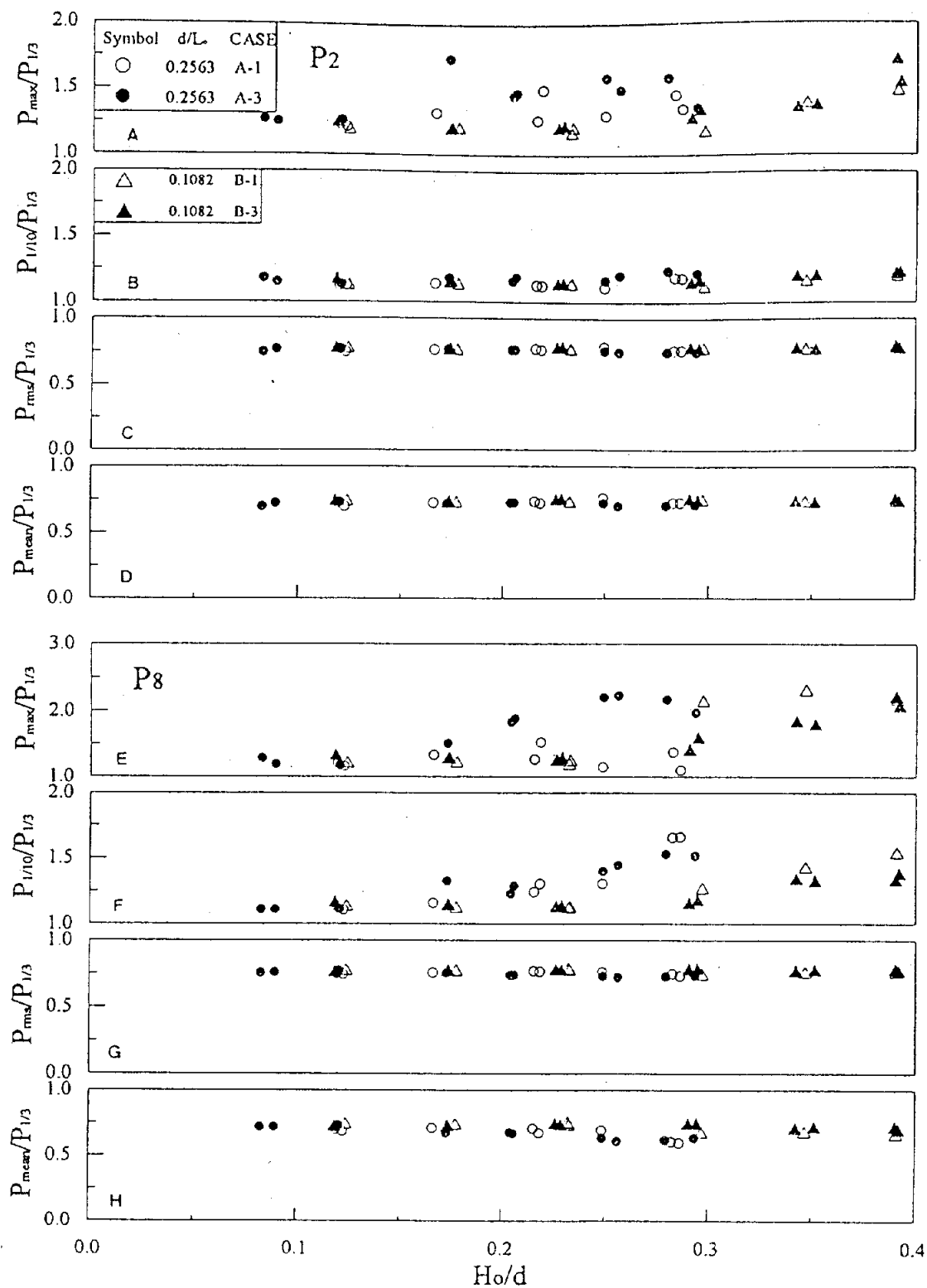


Fig-6 The statistical ratio value of wave pressures



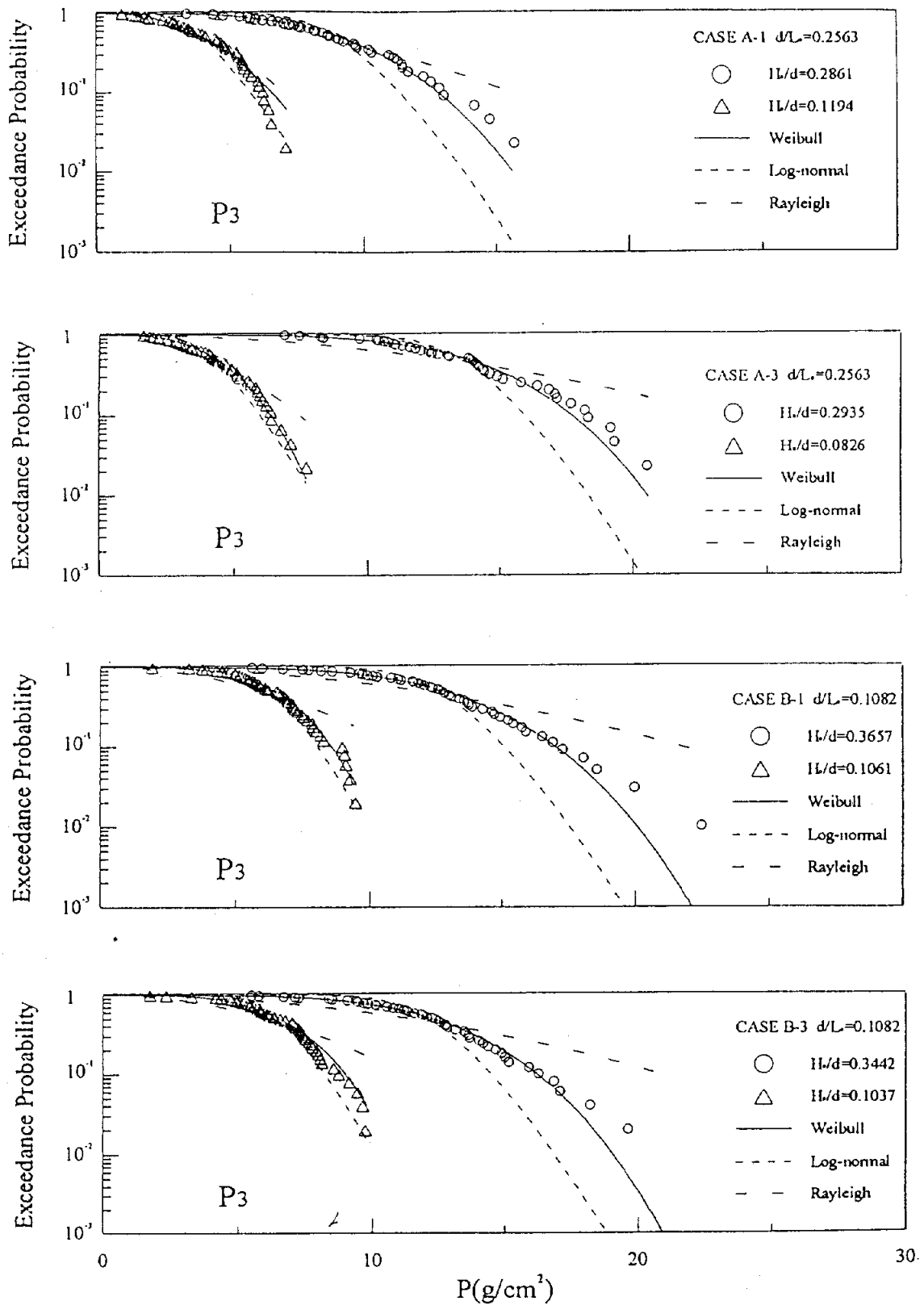


Fig-7 The exceedance probability distribution of wave pressures

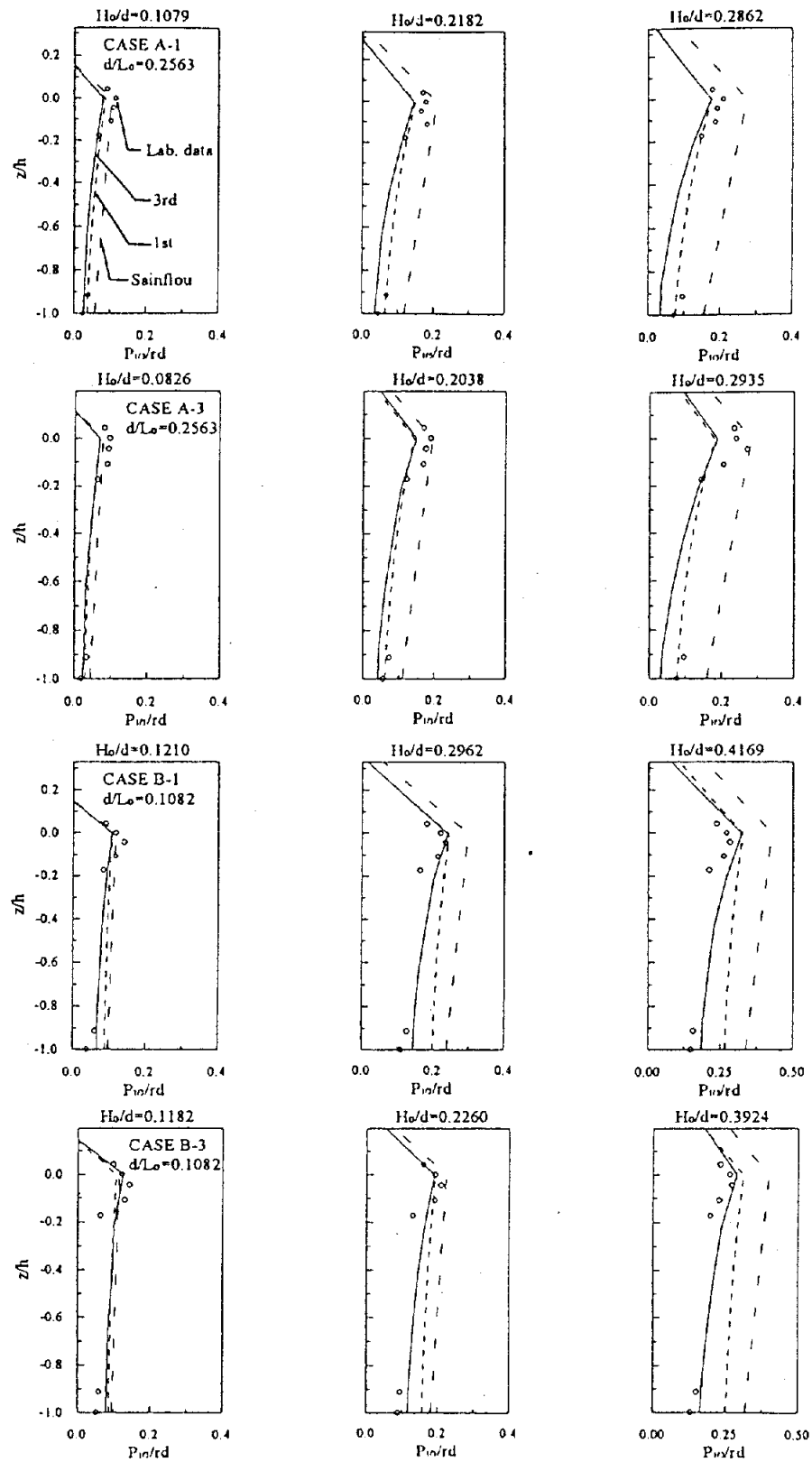


Fig-8 Pressures distribution on the front face of caisson

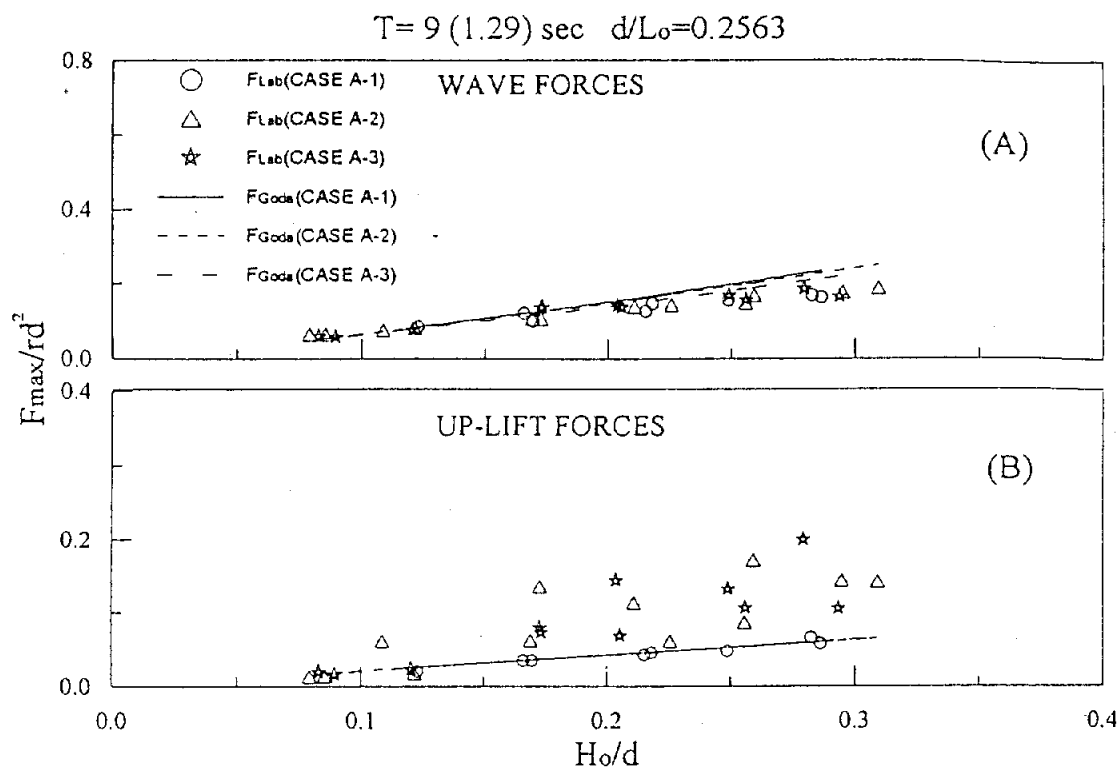


Fig-9 The comparison of  $F_{Goda}$  and  $F_{Lab}$

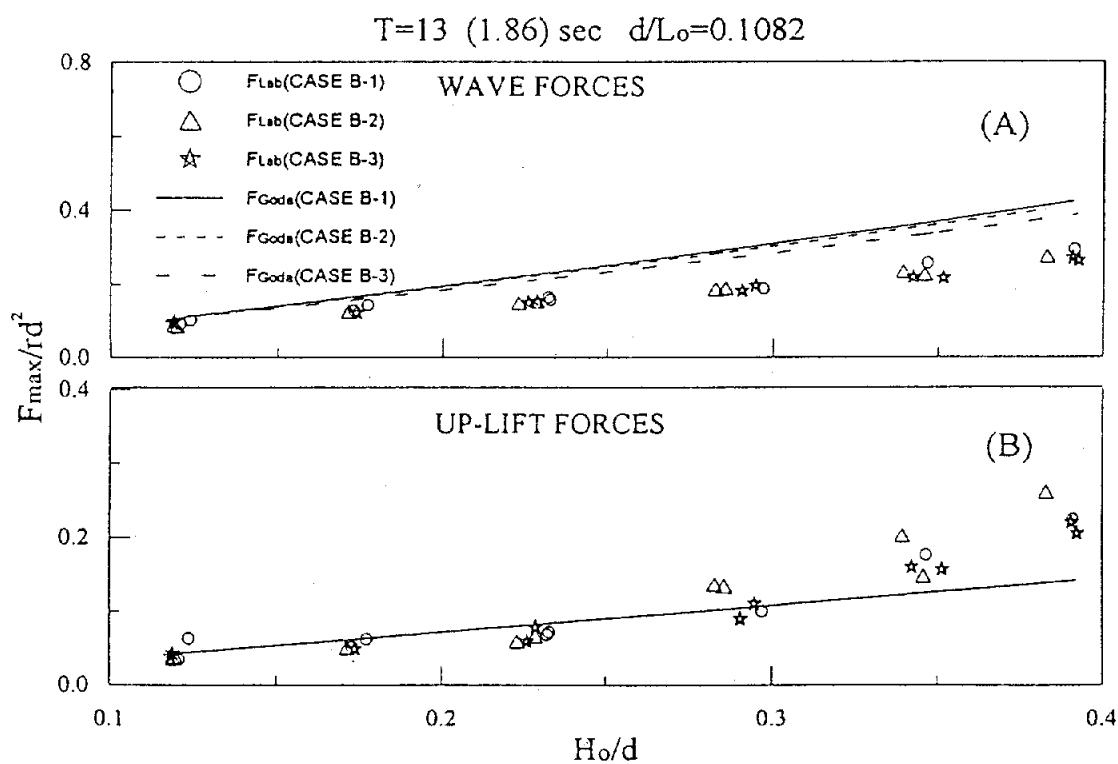


Fig-10 The comparison of  $F_{Goda}$  and  $F_{Lab}$

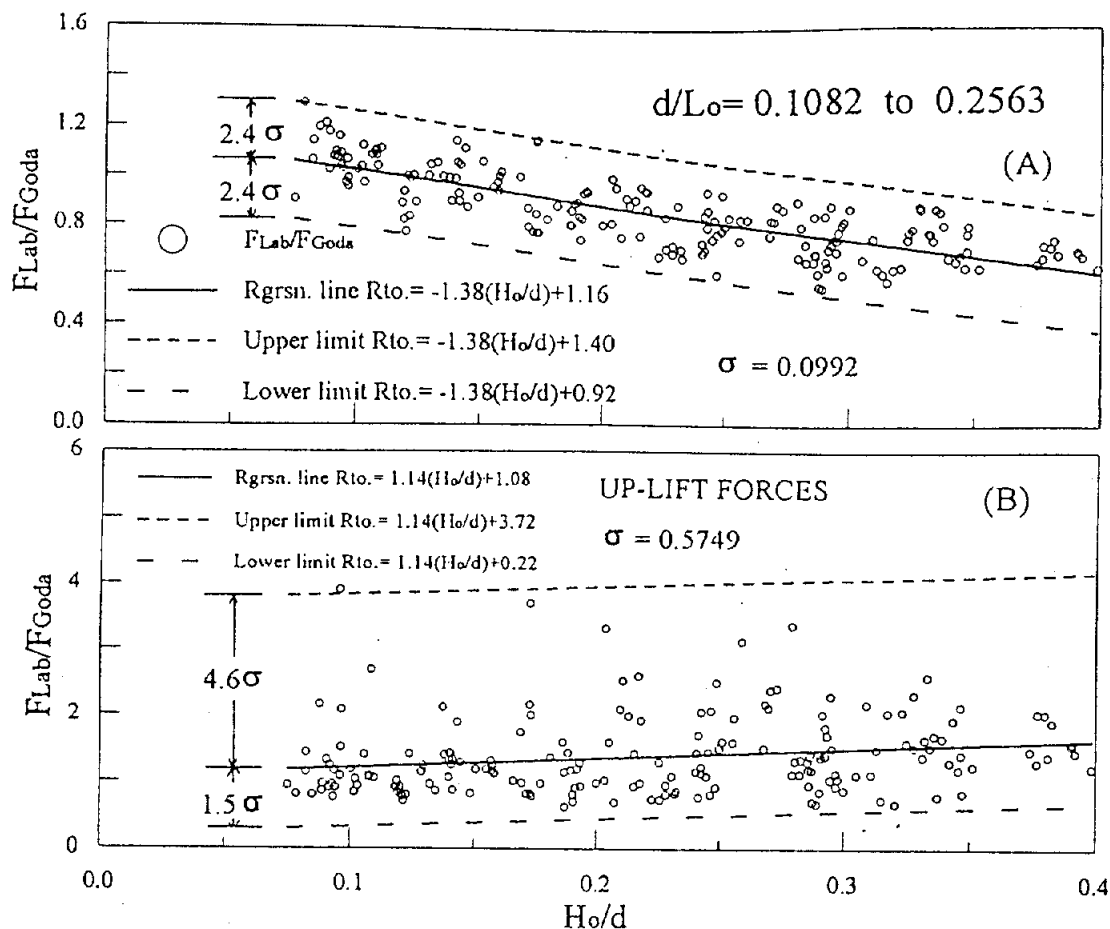


Fig-11 The relationship between  $F_{Lab}/F_{Goda}$  and  $H_0/d$

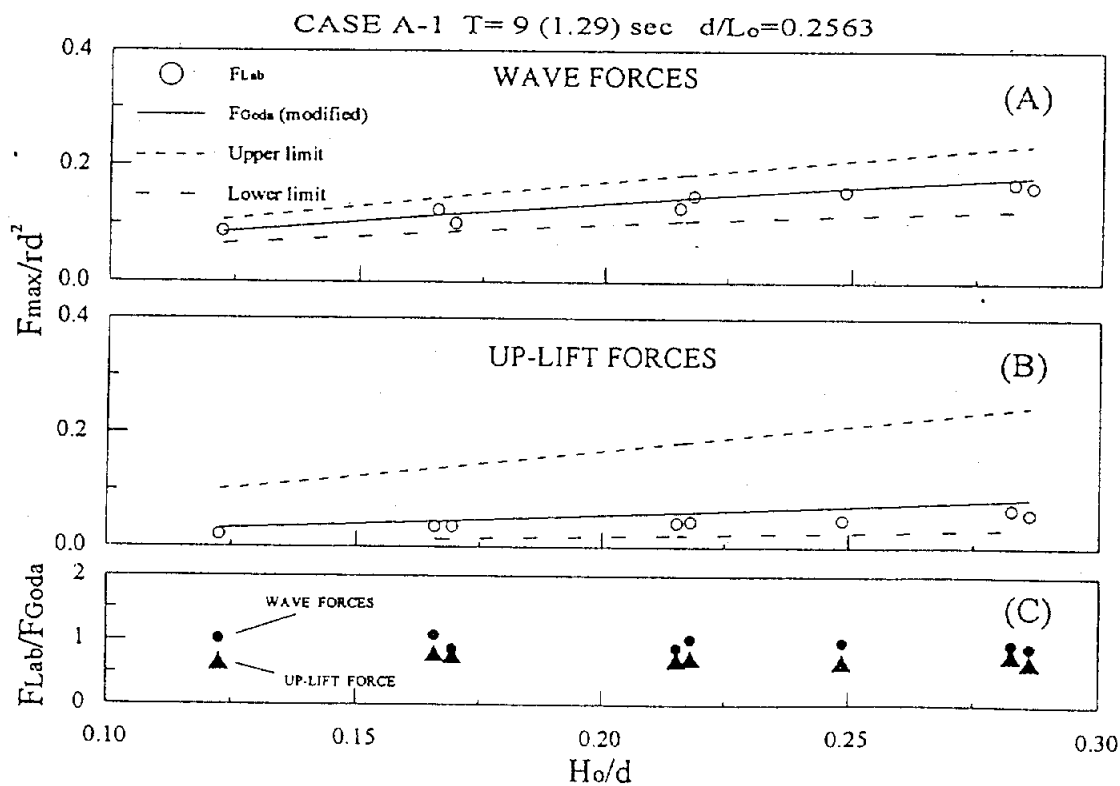


Fig-12 The comparison of experimental and modified data

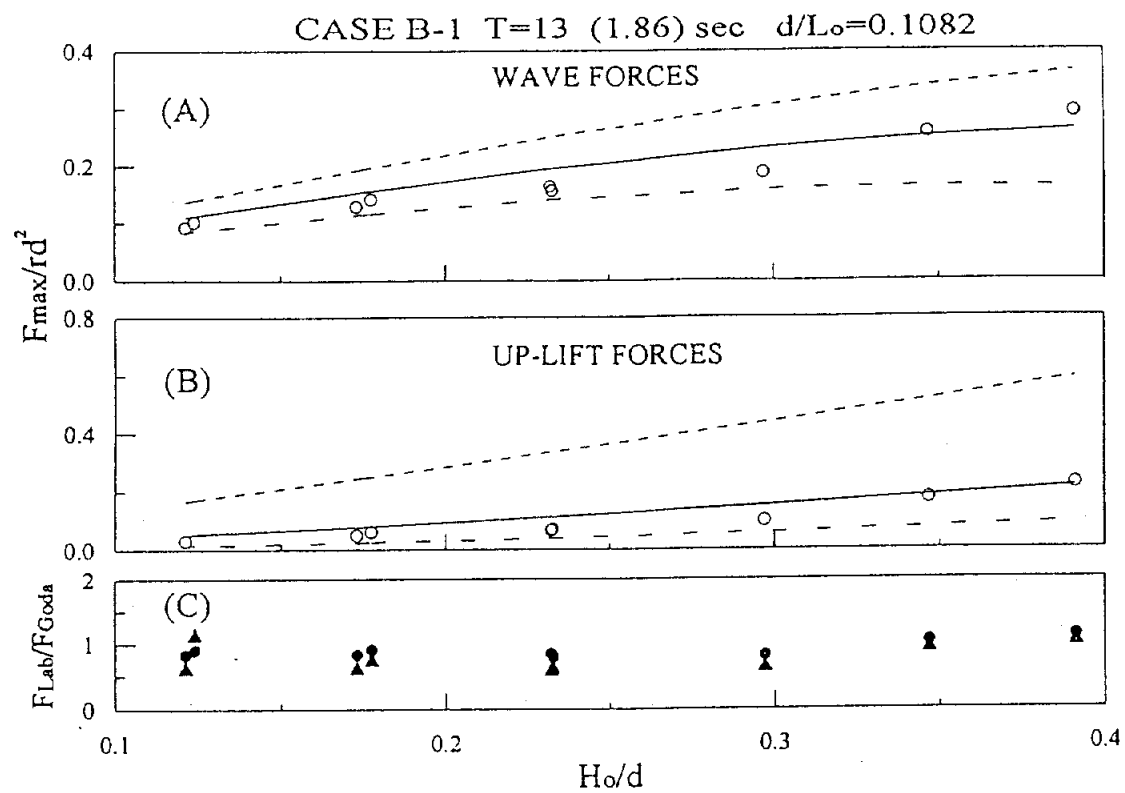


Fig-13 The comparison of experimental and modified data

# Study of Deep Water Breakwater Design Factors

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## ABSTRACT

The discussions in this study indicate that as a breakwater gradually enters a deep water area, the stabilizing condition during an earthquake will determine the size of the cross section of the breakwater. The control of the stabilizing condition will gradually shift from the sliding force to the bottom slab reverse force. In addition, the deep water area leads to a dramatic increase of the bottom slab reverse force, and the deterioration of the leveling precision of the rubble mound surface, which results in increased but uneven force imposed on the bottom slab of the caisson. Therefore, the design should be directed to the development and selection of new types of caisson to reduce the impact of external force, and the weight of the caisson. This way in turn reduce the bottom slab reverse force, and minimize the cross section of the caisson. As for the bottom slab design in relation to the leveling of rough surface, the load coefficient can be used to indicate the random reverse force, and achieve the goal of economic design.

**Key Words :** bottom slab reverse force, bearing capacity of the rubble mound,  
leveling precision, impact breaking wave force, liquefaction,  
earthquake reaction analysis

## 1. INTRODUCTION

The operation of a deep water harbor requires a large hinterland, which can provide warehouse facilities for the storage and allocation of various materials. If possible, the hinterland should also be large enough to include land for a seaside industrial zone, or even an airport to carry out the utmost function of a deep water, and increase its investment return. However, due to the limited space in Taiwan at the locations suitable for the development of a deep water harbor, the land on shore has mostly been developed. It is extremely difficult to acquire a large

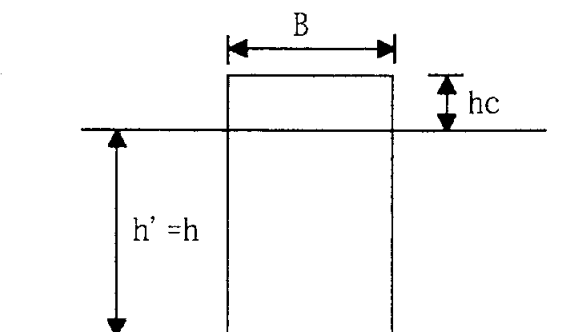
piece of land for the development of a large scale harbor and an airport. As a result, the most applicable way is to acquire land mass from the sea. In this case, the success and economic feasibility of this type of engineering becomes the key of the entire construction of a deep water harbor.

One of the key problems in the project of a deep water harbor is the design and construction of the deep water structure. Most of the deep water breakwater made by Sines in Portugal was destroyed in a single typhoon. This fact tells us that the design of the deep water structure is not a simple task. Other than the accurate consideration of various possible external forces, the economic aspect of the engineering, the feasibility and convenience of the construction must also be considered. Hence, it is a crucial subject for study. In a deep water context, the bottom of the upright breakwater is often subject to a very strong force, while the cross section of the rubble mound breakwater requires an immense cross section. Therefore, in general, a composite breakwater is more suitable for a deep water context, which is also the subject of this study.

## 2. EVALUATION OF THE DESIGN OF DEEP WATER BREAKWATER

### 2.1 Effects of a Deep Water Context on the Stabilizing Conditions of the Breakwater

The wave force acting on the Upright walls is generally calculated by the GODA(1974) formula. Here, the evaluation of the stabilization of a breakwater is based on the following assumptions. For example, since this evaluation focuses on the variation of the size of caisson, it is assumed, for the purpose of simplifying the computation, that the caisson is placed directly on the sea bottom. The various design conditions and the size of the cross section is shown in the following illustration:



- In this illustration,  $h$ : water depth in front of the breakwater  
 $hc$ : height of the caisson above the still water level  
 (here it is designated at 5 m)  
 $h'$ : water depth of the bottom of the caisson  
 $B$ : width of caisson

Design conditions:

- $\theta$ : incoming angle of the wave ( $\theta=0$ )  
 $S$ : Slope of sea bottom ( $S=1/\infty$ )

$H_{\max}$  : wave height used in design ( $H_{\max}=15\text{m}$ )

$T$  : wave period ( $T=12\text{sec}$ )

$\gamma$  : the average unit weight of the caisson

$\gamma_w$  : the unit weight of the sea water

Generally, a caisson must satisfy the following conditions to maintain its stability:

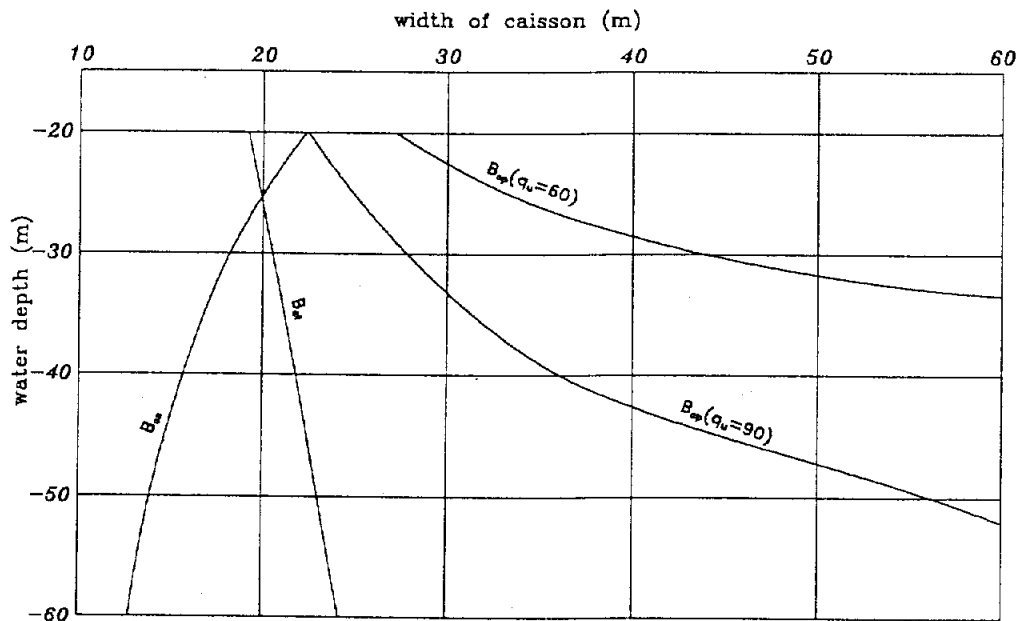
Sliding safety factor  $\geq 1.2$

Overturing safety factor  $\geq 1.2$  for wave force

$\geq 1.1$  for seismic force

Bottom slab reverse force  $\leq$  the allowable bottom slab reverse force  $q_u$

Let the caisson width determined by the sliding stability be  $B_{cs}$ , the caisson width determined by the overturning stability be  $B_{ct}$ , and the caisson width determined by bottom slab reverse force be  $B_{cp}$ . After computation, the relationships between each of the aforementioned width and the depth are shown in Figure 1.



**Figure 1** Relationships Between Each of the Aforementioned Width and the Depth

Figure 1 indicates that  $B_{ct}$  and  $B_{cp}$  tend to increase along with increasing depth, especially for  $B_{cp}$ . Although the required caisson width must satisfy all of the three stability conditions at the same time, the caisson width is mostly determined by the sliding stability with a smaller depth. As the depth increases, the caisson width tends to be determined by the bottom slab reverse force. Therefore, if the bottom slab reverse force could be reduced, the required caisson width can also be reduced to form a more economical cross section.

According to the current Japanese design standards, the maximum value of the allowable bottom slab reverse force is  $60\text{t/m}^2$ . To understand the effects of the allowable bottom slab reverse force on the breakwater body width, we used  $q_u=90\text{ t/m}^2$  to calculate the required caisson width. Figure 1 shows that the caisson width determined by  $q_u=90\text{ t/m}^2$  is smaller than



that determined by  $q_u=60 \text{ t/m}^2$ . Since the value of  $60 \text{ t/m}^2$  of the allowable bottom slab reverse force is not a reasonable value, further evaluation is necessary in a deep water context. This is also an important factor in the pursuit of economy in building a deep water breakwater.

## 2.2 Changes in the Design Conditions Concerning External Forces Resulted in a Deep Water Context

The wave force and seismic force are the primary external force acting on the breakwater. This section discusses which of the two controls the cross section of the structure as the depth changes.

The evaluation results of the calculation is shown Figure 2. The wave condition is  $H_{\max}=15 \text{ m}$ , and the seismic coefficient  $K=0.15$  and  $K=0.25$ .

Figure 2 indicates that when the caisson is placed above  $-20 \text{ m}$ , the required width under the

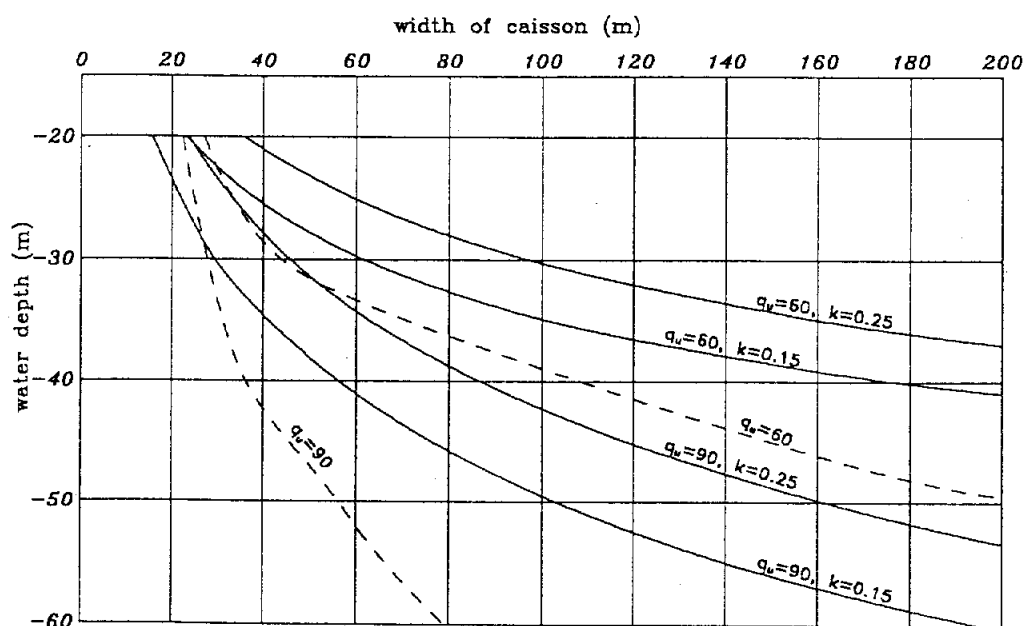


Figure 2 Comparison of Caisson Width Determined by Wave Force and Seismic Force

force of waves is always larger than that under an earthquake, regardless of the condition of the acceptable bottom slab reverse force. However, as the position of the caisson gets deeper than  $-20 \text{ m}$ , the seismic force gradually increases to surpass the wave force, and becomes the major force in deciding the cross section width of the caisson, especially when  $K=0.25$ , which also indicates a large seismic coefficient. The reason lies in that the required caisson width increases at a faster rate under an earthquake than in other circumstances. That is to say that as the depth increases, the stabilizing condition under an earthquake gradually control the cross section width of the caisson.

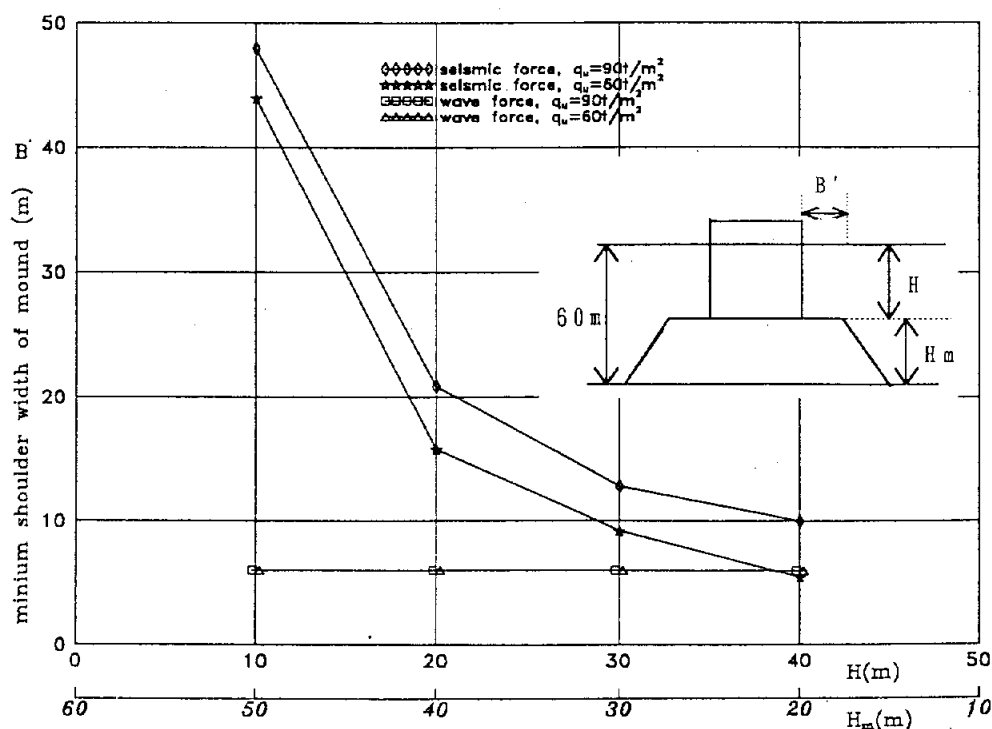
## 2.3 Evaluation of the Stabilizing Conditions of the Rubble Mound

The previous section deals with the caisson in the a composite breakwater. The tendency of an increasing cross section as a result of the increasing depth is examined. However, the evaluation of the economics of a composite breakwater should include the rubble mound for a comprehensive consideration, which is the subject of this section.

Here, the examination primarily discusses the changes of the height ratio of the caisson and the height of the rubble mound in a composite breakwater.

Let the depth be -60 m. Based on the relationship between the depth and the required caisson under the wave force and seismic force as shown in Figure 2, place the caisson respectively at the positions of -20 m, -30 m, -40 m and -50 m. Obtain the minimum rubble mound width that can satisfy the stabilizing condition by mean of the Katayama et. method(1972). Then examine the effects of different acceptable bottom slab reverse forces on the rubble mound . That is, find out the relationship between the acceptable bottom slab reverse force and the front shoulder width of the rubble mound that meets the stabilizing condition when the allowable reverse force  $q_u=60 \text{ t/m}^2$  and when  $q_u=90 \text{ t/m}^2$ . In this examination, for the soil conditions, let the angle of internal friction of rubble  $\phi=40^\circ$  ,and the sea bottom  $\phi=30^\circ$ .

The relationship between the depth and the minimum front shoulder width of the rubble mound that meets the stabilizing condition is shown in Figure 3. Under the wave force, due to the smaller horizontal force, as far as the depth and the allowable bottom slab reverse force are concerned, the current design of a rubble mound shoulder width of 6 m already meets the requirement of the bearing capacity. But under the seismic force, the required rubble mound front shoulder width has to be larger than that under the wave force due to the large horizontal force. The front shoulder width also varies according to the depth of the caisson position and the conditions of the allowable bottom slab reverse force.



**Figure 3** Relationships Between the Depth and the Minimum Front Shoulder Width of the Rubble Mound

## **2.4 Bearing Capacity of Rubble Mound**

There are a great number of varieties and types of harbor structure. The composite caisson breakwater in a deep water context can be considered as the representative of gravity type structure. In the design of a gravity type structure, the bearing capacity of foundation is the most important of all.

In general, the empirical value of the designated allowable bottom slab reverse force is used for the design of a gravity type structure due to the fact that the caisson is mostly placed on the rubble mound, and that the bearing capacity analysis of the rubble mound is yet to be established. For example, in Japan, the design standards for harbors set the maximum limit at  $60 \text{ t/m}^2$ . If one tries to use any bottom slab reverse force under the allowable value, the caisson width required by the deep water caisson becomes, in reality, a cross section that can not allow construction. The value of this allowable bottom slab reverse force is derived from the past experiences of the performance of structures. It provides a simple and convenient way for examining the bearing capacity. But for the more recent model or larger structures, there is yet to be the actual performance of large scale structure for our reference. As a result, the applicability of the limit of the allowable bottom slab reverse force needs to be further investigated.

From the discussions in sections 2.1 and 2.2, we know that as the depth of the breakwater increases, the sliding and overturning stability are still beyond the minimum requirement. The designation and limit of the allowable bottom slab reverse force are determined by the various conditions of the structure. Therefore, the rubble mound bearing capacity and the limit on the allowable bottom slab reverse force need to be further studied.

## **2.5 Design of the Bottom Slab of the Caisson**

In the construction of a harbor structure, the task of rubble leveling was usually done manually by divers. For the top surface (the contact surface with the caisson) of rubble mound base in a composite breakwater, the leveling precision is usually designated at  $\pm 5\text{cm}$ . The leveling operations are very time consuming because of the various sizes of the rubbles. However, when the depth of a breakwater exceed -20 m or even -30 m and above, the diving operations on the rubble mound becomes very difficult. The operation time of divers is shortened. Besides, there is a lack of qualified divers who can handle deep water operations. As a result, manual leveling operations become very difficult. It is thus absolutely necessary to develop mechanical leveling operations, and relax the leveling precision requirement. The applicability of the current design of the caisson structures placed on top of the rubble mound should also be examined and evaluated.

## **3. DESIGN CONDITIONS OF DEEP WATER BREAKWATER**

### **3.1 Investigation of Design Conditions**

(1) The condition of ocean climate

A. Typhoon waves

In the offshore area around Taiwan, actual data about the typhoon waves are insufficient. In general, numerical computation are the major data source in the designated wave of breakwater. In the past, for shallow offshore areas, the designated wave height facing the breakwater is restricted by the depth. Hence, the computation error of the deep water wave has limited effects on the design of the breakwater cross section. However, for a deep water breakwater, the computation of the deep water designated wave has a great impact on the design of the breakwater cross section. Therefore, the investigation of typhoon waves should be conducted before the deep water designated wave is decided. The computation results of the typhoon waves should be examined to determine the accurate and appropriate designated wave.

#### B. Regular waves

The construction cost of a deep water breakwater is immense. Hence, the planing of the breakwater layout should not only consider the calmness within the harbor, but also the reduction of construction cost. The construction of a deep water breakwater is an immense project, which must also endure rough ocean climate conditions. Hence, it is absolutely necessary to study how to carry out the construction in a large scale and fast way. To satisfy the various criteria in question, the long term observation of the waves at the planned location must be conducted to accumulate the actual data of typhoon waves for the purpose of reference during the planning, design and construction.

#### C. Tide line, tidal currents, etc.

To resolve the environmental impact of the surrounding offshore area on the deep water breakwater during and after the construction, investigations concerning the basic data of tide, tidal currents, water temperature, and salinity should be conducted regularly.

#### (2) The soil condition

It is extremely difficult in terms of technologies to conduct soil investigations in a deep water context. It requires highly advanced technologies and very high cost. Hence, the density and items of the investigation have to be determined according to such comprehensive parameters as the surrounding sea bottom typography, the amount of sediment washed down from nearby rivers, and the longshore currents.

A deep water composite breakwater is a large piece of structure with considerable weight. Therefore, the bearing capacity of the sea bottom is a very important point in the design. It is thus necessary to investigate the static characteristics concerning the soil of the sea bottom. Since a deep water breakwater has a tall rubble mound base, the resistance of the breakwater to earthquake becomes another important factor in the design. It is necessary to investigate the kinematics characteristics of the sea bottom for the design of earthquake resistance.

#### (3) Earthquake investigation

The structural behavior during an earthquake is determined by the foundation, the kinematics characteristics of the structure, and the nature of the earthquake. Therefore, the earthquake design must be cautiously selected during the design of earthquake resistance.

On the other hand, the earthquake records, including the earthquake activities in the area and the characteristics of the earthquakes, must be investigated to select a specific target for

earthquake design. The selection of the targeted earthquake for design has to take into consideration the oscillation characteristics, the importance, and the life of the breakwater.

### **3.2 Investigation of Construction Materials**

The rubble mound of a deep water composite breakwater requires a large amount of rubbles. The difficulty of acquisition and the price have an absolute impact on deciding the thickness of the rubble mound of the breakwater. Therefore, the source, quantity, quality, delivery and price of rubbles must be carefully investigated.

### **3.3 Investigation of Construction Techniques**

#### **(1) The construction and positioning of the large caisson**

During the construction of the caisson, the various details such as the location and scale of the construction area, the construction method of the caisson, the method of moving the caisson, the method of water intake and the number of caisson to be constructed must be examined. As for the construction of the large caisson, one should take special care to consider the way to move the completed caisson and the water intake, other than the production of the caisson.

#### **(2) The construction method of the rubble mound**

The following problems should be considered in building a rubble mound in deep water:

##### **A. How to prevent the rubbles from scattering outside the designated cross section:**

The rubble mound basis of a deep water composite breakwater is usually taller than other types of breakwater. Therefore, it is very important to apply a rubble mound building method with a large quantity of rubbles, speed and accuracy. At present, the rubble ship operation is widely used. However, the scattering and accumulation of rubbles vary a great deal according to the ship type, rubble mound cross section and placement depth. Therefore, the design must fully control the scattering and accumulation characteristics of the rubbles so as to choose the most appropriate rubble placement method and placement sequence. In particular, there is a limit to the slope formed by the rubble mound in deep water. One has to take special care about the method of placing the rubbles.

B. The degrees of levelness of the top surface after the rubble mound is completed affects the efficiency and leveling precision of the leveling operations. Besides, the amount of settlement and uneven settlement of the caisson after the caisson is positioned has to be prevented. It is thus a worth subject to study how to place the rubbles so as to build a dense rubble mound.

##### **C. The establishment of construction management**

To solve the above problems, the rubble placement must be conducted according to the rubble placement plan in addition to the fully understanding of the characteristics of the rubble ship. Hence, an accurate construction management is indispensable. In addition, to prevent the formation of a wasteful cross section, the speedy and precise guidance of the rubble ship to the position for rubble placement, and the maintenance of the ship in the same position during the rubble placement process should be evaluated.

#### **(3) The leveling method of the rubble mound**

The leveling operation of the rubble mound is the latest type of operation to be done by machines in the construction of a harbor. Almost all of the operations depend on divers. It is easy to see the construction conditions and difficulty. As a result, the deeper the position of the leveling operation, the lower the efficiency of the operation. At the same time, there are also safety concerns. But in recent years, the development and improvement of mechanical leveling has made deep water mechanical leveling more feasible. Therefore, it is necessary to examine the operation conditions, suitable depth, leveling precision and operation cost of the leveling machine during the design.

### **3.4 Cross Section Type of the Breakwater**

In general, a breakwater should possess the following functions:

- good resistance to waves (wave height transition feature)
- complete safety (resistance to waves and resistance to earthquake)
- minimum impact on the surrounding environment (water exchange reflection waves, erosion and accretion etc.)
- low construction cost (building cost, maintenance cost)
- nature of construction (inland operation, offshore operation)

Certainly, it is impossible to build a breakwater that meets all of the above functions. Therefore, the current practice is to plan or design, based on the current conditions, something that can satisfy part of the aforementioned functions. The selection of the type of cross section must decide the priorities of the various functions.

### **3.5 Design of Wave Resistance**

The wave resistance design of a deep water composite breakwater may follow the design flow chart and design standards of a shallow water breakwater. But the following items must be considered:

#### **(1) Impact breaking wave force**

The occurrence of impact breaking wave force is related to the wave conditions as well as the shape of the rubble mound. Since the conditions for the occurrence of impact breaking force are very complicated, which is especially true for a deep water composite breakwater with tall rubble mound, the impact breaking wave force has to be examined according to the model test.

#### **(2) The stability of the upright portion**

In the case that the cross section of a deep water composite breakwater may invite impact breaking wave force, the model test must be conducted to obtain the intensity of the wave pressure and the total force acting on the upright portion. If the experiment in question confirms the impact breaking wave force, the cross section that maintains the stability of the upright portion will become extremely uneconomical. It is better to change the type of the cross section, and adjust the height of the rubble mound to overcome the problem.

#### **(3) The stability of the rubble mound**

At present, there are a certain formula to calculate the required weight of the protection blocks for the breakwater and the covering materials around the depth of -20 m. But there is a great

difference among the weight derived from these formulas. As far as a deep water composite breakwater is concerned, there are still a number of questions about the formulas currently in use. Therefore, it is necessary to examine the stability of the rubble mound with the model test.

### 3.6 Design of Earthquake Resistance

The design of earthquake resistance of a deep water composite breakwater must consider such factors as the frequency of earthquake activities at the site of breakwater construction, the life, importance and oscillation characteristics of the breakwater. The following items should be examined:

- analysis of the potential of liquefaction of the sea bottom
- examination of the allowable bearing capacity and sliding behavior of the rubble mound
- examination of the sliding, overturning and bottom slab reverse force of the caisson
- examination of the settlement of the breakwater

The earthquake resistance design of harbor facilities adopt the seismic coefficient method. But for a structure as tall as a deep water breakwater, the distributions of the horizontal seismic coefficient (acceleration) at various height are different. This has been proved from the observation of intense earthquakes and experiment of model oscillation. Therefore, the earthquake resistance design must be made according to a modified seismic coefficient method. When the modified seismic coefficient method is used, the analysis of earthquake reactions must be conducted to obtain the seismic coefficient in action at different height and direction.

At present, while conducting the evaluation of earthquake resistance, one still has to pay attention to the following problems:

- (1) Based on the prediction method of the liquefaction of sea bottom proposed by the Japanese harbor technical standards, no matter which method is used, the analysis should be made with the reaction acceleration or the maximum shear stress of the sea bottom during the earthquake. But for a deep water composite breakwater with the complicated structural system of caisson-rubble mound sea bottom, whether the above method is applicable requires further examination. Hence, it is necessary to study the liquefaction potential of the sea bottom beneath the structure.
- (2) As for the examination of the stability of the rubble mound base, there is yet to be completing basic data on the kinematics characteristics of the materials of the rubble mound. Hence, the applicability of related designs will require a considerable amount of time to verify.
- (3) The formula for calculating the extent of sliding, the acceptable amount of sliding and the allowable bottom slab reverse force caused by an earthquake still require further examination and study.
- (4) The compression and settlement of the rubble mound of a breakwater during an earthquake are still in the initial stage of research. The examination method is yet to be established.

### 3.7 Design of the Caisson Slab

Due to the deterioration of leveling precision of the deep water breakwater, the impact on the caisson should be examined according to the following methods:

- conduct calculation with a numerical simulation
- conduct an experiment with a hydraulic model
- conduct an experiment with a real object

During the calculation with a numerical simulation, the following factors should be noted:

- (1) As the leveling precision deteriorates, one should use random numbers to simulate the irregular distribution of the slab reverse force, and use the finite element method to conduct slab analysis.
- (2) Since the rubbles are distributed irregularly without any specific shape, there should be a variety of simulated calculations. Then the statistical concept can be applied to evaluate the impact on the design of the bottom slab.

#### 4. CONCLUSION

- (1) Once in a deep water context, the stabilizing conditions of the caisson gradually shifts from the control by sliding to the control by the bottom slab reverse force. The design of external force also tends to be controlled by the seismic force. As a result, during the design, the development and choice of new type of caisson should be achieved to reduce the impact of external force and the weight of the caisson, which may further reduce the bottom slab reverse force and the caisson cross section.
- (2) In a deep water context, the design of external force of the breakwater gradually shifts from the control by the wave force to the control of the seismic force. Therefore, the design of earthquake resistance must be conducted. The evaluation items include the analysis of the liquefaction potential of the sea bottom, the allowable bearing capacity and sliding behavior evaluation of the rubble mound, the sliding, overturning and bottom slab reverse force of the caisson, and the settlement of the breakwater.
- (3) To study the cross section of a deep water composite breakwater, one must fully understand the functions of wave prevention, safety, environmental protection, construction cost and nature of construction, and decide the priorities of these targets.
- (4) During the planning and design stage of a deep water breakwater, one must investigate the limit of related current construction techniques, the direction of technological development, and estimate the construction techniques to come on a mid-term and long term basis.
- (5) The inclusion of the concept of probability during design has become a design trend in recent years. Therefore, we suggest that the design of the bottom slab with rough leveling be evaluated by the concept of statistics so as to use the loading coefficient to present the random reverse force in meeting the purpose of economical design.



## 5. SUGGESTIONS

- (1) Once in a deep water area, the allowable bottom slab reverse force has a great impact on the required width of the caisson body, which further affects the cost of the construction. The specification of the value of the allowable bottom slab reverse force should be further evaluated.
- (2) There is very little research available on the liquefaction potential of the sea bottom soil beneath the structure. It is a subject worthy of further study.
- (3) The research and development of machines suitable for rubble placing and leveling in a deep water area should be conducted as soon as possible.

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