Progress Report

April 30, 2007

PROPOSAL TO THE FEDERAL HIGHWAY ADMINISTRATION

TASK ORDER DTFH61-06-T-70006

FOR THE DEVELOPMENT OF
GUIDE SPECIFICATIONS FOR BRIDGES VULNERABLE TO COASTAL STORMS
AND
HANDBOOK OF RETROFIT OPTIONS FOR BRIDGES VULNERABLE TO COASTAL STORMS

by

Modjeski and Masters, Inc.

with

Moffatt and Nichol, Inc.
Ocean Engineering Associates, Inc.
D'Appolonia, Inc.
Dr. Dennis R. Mertz

INTRODUCTION

We received Notice to Proceed on this Work Order on August 14, 2006.

This report covers work done in April, 2007. During this month the team held several conference calls and development presentation information in preparation for a meeting with the Task Force Members with ocean/hydraulics interest.

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TASK 1 – MEETINGS

A meeting with the task force members with ocean/hydraulics interests and the FHWA representatives took place in Baltimore on April 19, 2007. The minutes of the April 19th meeting are attached as Attachment A.

A meeting with the full Task Force is scheduled for June 12 and 13th in Raleigh, North Carolina.

TASK 2 – REVIEW, SUMMARIZE, AND AUGMENT LITERATURE

Work on Task 2 is essentially complete. Some refinement may be incorporated when transferring the information developed in this task to the final report of the project.

TASK 3 – REVIEW AND SUPPLEMENT ONGOING FORCE STUDIES

Dr. Sheppard continued his work on testing the bridge model in the lab. Mr. Shelden continued his work on developing results for different cases using Wallingford's and Douglas's methods and compare these results to the lab tests. A significant portion of the above work was presented and discussed during the April 19th meeting. See Attachment A for more details on the work conducted up to the time of the meeting. Work continued after the meeting and the following is a brief description of the work conducted between April 19 and April 30th. Some of this work was conducted in response to comments received during the meeting.

- Dr. Sheppard continued his work on developing the graphs for the design forces. To improve the accuracy, new drag and inertia coefficients that correspond to UF lab results are being developed and the design graphs will follow. At the time of the meeting it appeared that a large number of graphs may be necessary. It now appears that curve fitting may be possible. This will result in representing each curve with a simple equation, probably as simple as a quadratic equation. Instead of all the graphs, a table listing the constants for the equations representing the curves will replace the actual graphs. Dr. Sheppard is investigating this approach which will result in a significant reduction in the size of the specifications while simplifying its application. It should be noted that the vertical wave force data plots presented by Dr. Sheppard at the April 19th meeting do not include the slamming force as was stated. These data are for the quasi-static component only. Curves (and equations) with the total forces will be provided in the final submission.

- Mr. Shelden continued his work on comparing results using Wallingford method to those developed using Modified Kaplan. Earlier comparisons indicated that, generally, Modified Kaplan produced smaller force magnitudes. This caused some concern as both methods supposedly were developed based on to lab test results.

  Additional calculations for the Wallingford method were performed using a different set of coefficients presented in their reports. These calculations resulted in forces of a similar magnitude to the Modified Kaplan method and lab results. It is evident that the application of the Wallingford method to typical bridge structures is complicated by the nature of their tests. These tests were for a different type of structure configuration and the measured results were for individual structural elements, not the global loads on the structure. Extrapolating these structural element loads to global loads is highly dependent on the structural configuration and the wavelength to structure width ratio. This further reinforces the decision to pursue the modified Kaplan method for use in the guide specifications.

- Some work on determining the area of opening required to vent the air from compartments between girders was conducted. So far, the work does not consider the compressibility of the air. Further refinements to consider the compressibility are underway.

TASK 4 – COMPILE AND CATALOG RETROFIT OPTIONS

Work on Task 4 is essentially complete. Some refinement may be incorporated when transferring the information developed in this task to the final report of the project.

TASK 5 – PERFORM ANALYTICAL STUDY OF RETROFIT OPTIONS

No progress to-date. A proposal was submitted to the FHWA to reduce this task and divert resources to Tasks 3 and 6.

TASK 6 – DEVELOP A GUIDE SPECIFICATION AND A RETROFIT HANDBOOK FOR ADOPTION BY AASHTO

TASK 6A - GUIDE SPECIFICATION

We continue to expand and refine the draft of the specifications. We shared a draft of the 50% specifications with the BWTF during the April 19th meeting. At the time of the meeting, most areas were sufficiently developed for the 50% submission with the exception of the articles containing the wave force calculation equations for superstructures. We are working on the development of the latter articles.

TASK 6B - RETROFIT HANDBOOK

Work on the retrofit manual continued. An updated screening procedure is expected from OEA in the next few days. The updated procedure will be incorporated in the manual. A cost model is being developed for incorporation in the retrofit manual by Mike Knott of Moffatt and Nichol.

TASK 7 – DEVELOP FINAL REPORT AND RECOMMENDATIONS FOR FURTHER STUDIES

No progress

TASK 8 – PREPARE EXECUTIVE SUMMARY AND PRESENTATION MATERIALS

No progress

FUTURE WORK – NEXT MONTH

1. Continue working on the issues raised during the April 19 meeting
2. Develop the method of calculating wave forces on superstructure and the associated specifications provisions
3. Submit the 50% specifications and retrofit Manual

SCHEDULE

See attached schedule.

<u>SCHEDULE</u>

TASK	Date shown in Work Plan	PROPOSED COMPLETION DATES
Notice to Proceed	September 1, 2006	
Kickoff Meeting	December 4,5,6, 2006	
Task 2	December 15, 2006	Done
Task 3	December 15, 2006	May 31st, 2007
Task 4	January 26, 2007	Done
Task 5	On hold pending resolution of proposal to the FHWA	
Task 6 50% Draft Specification and Manual 90% Draft Specification and Manual 100% Draft Specification and Manual Interim Report Tasks 2 to 6	 February 15, 2007 May 31, 2007 August 15, 2007 July 15, 2007	 May 15, 2007 July 31, 2007 October 15, 2007 September 15, 2007
Task 7 Draft Final	 June 30, 2007 September 15, 2007	 August 31, 2007 November 15, 2007
Task 8 – Executive Summary Draft 4 to 6 page summary Final 4 to 6 page summary	 June 30, 2007 August 31, 2007	 August 31, 2007 October 31, 2007
Task 8 – 13 hour slides Draft Final	 November 30, 2007 January 31, 2008	 January 31, 2008 March 31, 2008

Attachment A

Minutes of the April 19 Meeting

Harrisburg, Pennsylvania
May 1, 2007

MEMORANDUM

TO: Modjeski and Masters, Inc.

RE: APRIL 19, 2007 MEETING MINUTES – DTFH61-06-T-70006

PN2560

The April 19, 2007, meeting of the above-captioned project was held in Moffatt Nichol office in Baltimore. The following were in attendance:

Wave Vulnerability Task Force

Greg R. Perfetti (NCDOT)
Rick Renna (FDOT)
Tom Everett (FHWA)
Joseph Krolak (FHWA)
Kornel Kerenyi (FHWA)
Firas Ibrahim (FHWA)

(Tony) Robert A. Dalrymple,	Johns Hopkins University
David L. Kriebel,	U.S. Naval Academy
Spencer Rogers,	North Carolina Sea Grant

Project Team

John Kulicki (M&M)
Wagdy Wassef (M&M)
Max Sheppard (OEA)
Jeff Shelden (M&N)

PRESENTATIONS AND DISCUSSIONS

- **Team Presentation by Dr. Kulicki, Dr. Sheppard and Mr. Shelden**

  The team presentation covered the following topics:
  - Introduction – Purpose of Meeting
  - UF Laboratory Tests
  - Introduction to calculations methods
    - Douglas Method
    - Wallingford Method with calculations example (Escambia Bay Bridge)
    - Modified Kaplan Method
  - Sample Dimensionless wave force plots
  - Comparisons Between Predicted and Measured
    - Laboratory data
    - Field data (Escambia Bay failure patterns)
    - Parametric study
  - Selection Criteria for calculation method to be incorporated
    - Relationship with experimental results
    - Prediction of failures in field
    - Theoretical completeness
    - Practicality
  - Is a Hybrid Method of calculations possible (Not recommended)
  - Recommended method
  - Future work plans for wave force
  - Further Confirming Studies
  - Wave Force input Parameters
  - Walk-through of draft specifications in-progress
  - Load Factor Modifiers Based On Met/Ocean Joint Probability
  - Code Calibration

  A copy of the presentation is attached as Attachment A.

- **Recommended Method of Wave Force Calculations**

  After presenting the information related to the wave force calculation methods, the project team recommended that the Modified Kaplan method be adopted by the specifications. The Task force members attending the meeting discussed this recommendation in a closed session and accepted it. The research team will proceed with the remaining work on the project taking this into consideration. The Task force cited the following as some of the reasons they agreed with the recommendation:
  - Physics well accounted for
  - Accounts for the ratio between the width of the structure and the wave length
  - Includes the upward and downward force distribution. This is manifested in the inclusion of the overturning moment in addition to the net vertical and horizontal force.

- Wave forces found to be sensitive to wave period. Wave period was included in the analysis.

- **Coordination with Work by Others**

  Anticipated and in-progress related work by other teams was discussed along with test sites that can be used to conduct further verification work. The following was identified:
  - Dan Cox in Oregon State University is conducting work thought to be related more to Tsunamis.   The FHWA (Tom Everett) will investigate if AASHTO can ask Oregon DOT to coordinate their work to address the issues raised by our team
  - OSU: the facilities are large and may be used to replicate some of UF tests using a larger model. The intent is to check scale effects
  - USAOCS (Army test center) has the largest wave test capabilities in the US

- **Requested Additional Items**

  The discussions during the meeting resulted in requesting the research team to consider the following items:
  - Prepare a table showing the size of orifice required to evacuate a certain amount of air in 1 second.
  - Investigate why Wallingford associates say that Modified Kaplan gives low forces
  - Investigate how model scale affect the air entrapment behavior
  - Conduct UF test of the model with railings and overhangs
  - Check the convergence of modified Kaplan and Wallingford methods for small width spans subjected to long period waves
  - Consider the possibility of separating the slamming forces and the quasi-static forces.
  - The possibility of varying the load factor based on the level of analysis used was discussed. It was suggested that it may be better to vary the loads instead of varying the load factor. The latter approach is closer to the approach used by other specifications. Also it is not clear if it will be possible to include the load modifier in the Monte Carlo simulation. The idea of using load modifiers needs further study.
  - The idea of a load modifier based on the possible angle of wind attack as dictated by the geometry of the body of water was discussed and was, generally, not welcomed. No alternatives were suggested.
  - Include bridge systems with flat superstructure bottom surface (e.g. voided slabs and adjacent box beams) in the design charts.
  - Check to see if Wallingford and Kaplan methods converge for specific cases, e.g. for long period waves.
  - Include wording in the specifications that allows owners to design small bridges without considering wave forces
  - Clarify definitions to eliminate differences in interpretation, e.g.:

        - Define what is included in “current”
        - Define Max Wave (1% or other measure) and period
        - Define Wave Crest, FEMA uses 0.7, SPM has a figure (Fig. II-8-14, page II-29), CEM has a new figure.
    - Add Diffraction Forces calculation method to Morrison Equations for horizontal forces on bridge substructures and indicate that the equations are applicable for large structures
    - The proposed specifications are based on 100 year events. The flood analyses are usually based on 50 year events. The implications of using different return period for the waves and floods should be taken into account.
    - How will the overturning moment will be included in the specifications?

- **Proposed Change in the Scope of Different Tasks of the Project**

  Dr. Kulicki proposed shifting some of the funds earmarked for Task 5 to other tasks. This will serve the purpose of further verifying the wave force calculation method and building confidence in the accuracy of the method to be used in the specifications. The task force requested that the project team submit a formal proposal of the changes in scope and budget to Mr. Bob Prior. Mr. Prior will review the proposal and discuss it with the task force before responding to the research team.

WAGDY G. WASSEF

Attachment A

Team Presentation During the 4-19-2007 Meeting

Storm Surge/Wave Forces on Bridge Decks

Outline

- Introduction – Purpose of Meeting (John)
- UF Laboratory Tests (Max)
  - Spans with girders
    - No overhangs or rails
    - Range of wave parameters and span locations
  - Data reduction and analysis
- Douglas Method (Jeff)
- Wallingford Method (Jeff)
- Modified Kaplan Method (Max)

Overbeek and Klabbers (2001)

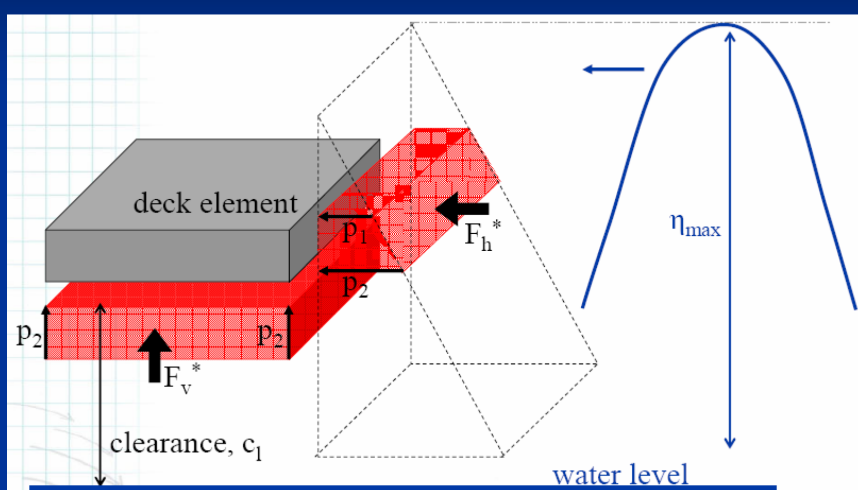
- Slowly Varying Pressure (P_{sv})

P_{sv} = 1.0 ρg (η_{crest} – d_c)
where η_{crest} = crest elevation above WL
d_c = deck bottom elevation

Peak Impact Pressure (P_i)

P_i = 1.5 ρg H_{max}
Where H_{max} = Maximum Wave Height

HR Wallingford – "Old Exponential" Method



Definition of "basic" wave forces F_h* and F_v*.

HR Wallingford – "Old Exponential" Method

$$F^*_v = \int_{b_w}\int_{b_l} p_2 \cdot dA \cong b_w \cdot b_l \cdot p_2 \qquad (1)$$

$$F^*_h = \int_{b_w}\int_{c_l}^{\eta_{max}} p_{hyd} \cdot dA = b_w \cdot (\eta_{max} - c_l) \cdot \frac{p_2}{2} \quad \text{for} \quad \eta_{max} \le c_l + b_h \qquad (2)$$

$$F^*_h = \int_{b_w}\int_{c_l}^{c_l+b_h} p_{hyd} \cdot dA = b_w \cdot b_h \cdot \frac{(p_1 + p_2)}{2} \quad \text{for} \quad \eta_{max} > c_l + b_h \qquad (3)$$

where

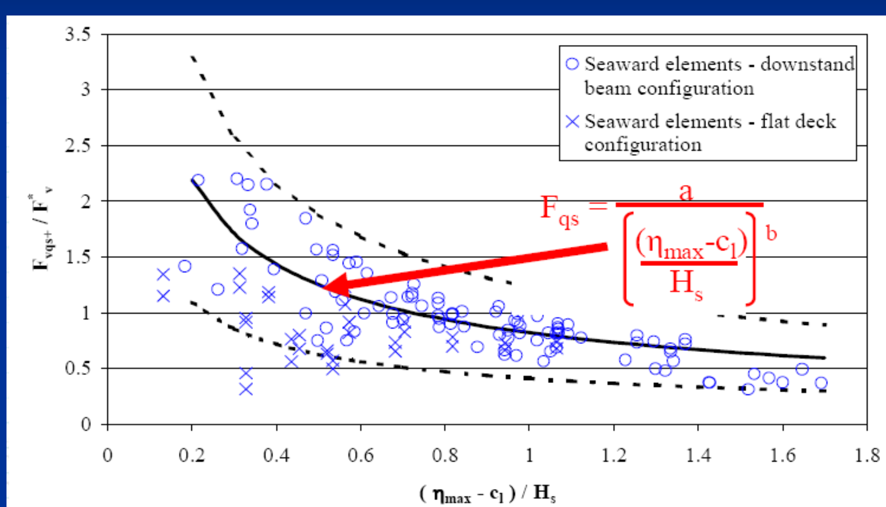
$$p_1 = [\eta_{max}\ \tilde{n}\ (b_h + c_l)] \cdot \rho g \qquad (4)$$

$$p_2 = (\eta_{max}\ \tilde{n}\ c_l) \cdot \rho g \qquad (5)$$

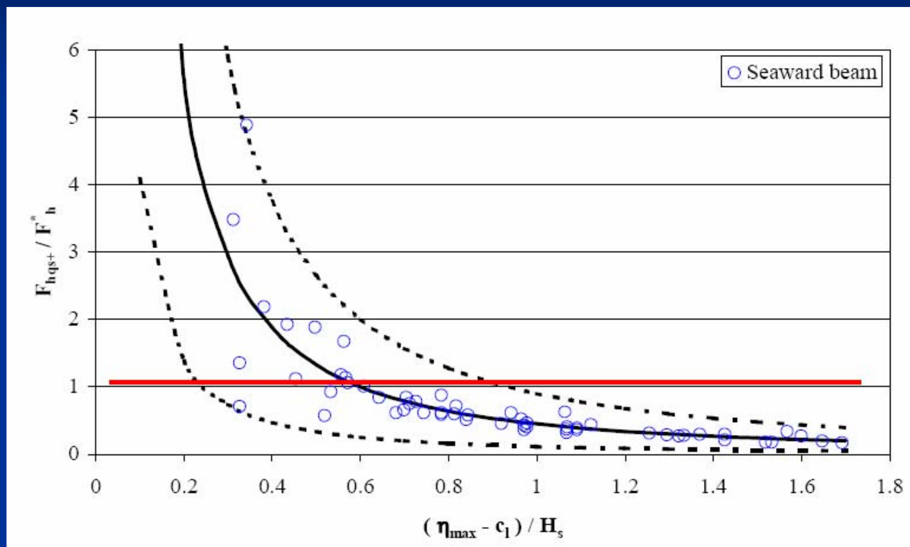
and

p_1, p_2	pressures at top and bottom of the element
b_w	element width (perpendicular to direction of wave attack)
b_h	element depth
b_l	element length (in direction of wave attack)
c_l	clearance (distance between soffit level and still water level, SWL)
η_{max}	maximum wave crest elevation (relative to SWL).

HR Wallingford – "Old Exponential" Method Quasi-Static



HR Wallingford – "Old Exponential" Method Quasi-Static



HR Wallingford – "Old Exponential" Method Quasi-Static

$$\frac{F_{qs}}{F^*} = \frac{a}{\left[\frac{(\eta_{\max} - c_l)}{H_s}\right]^b}$$

where

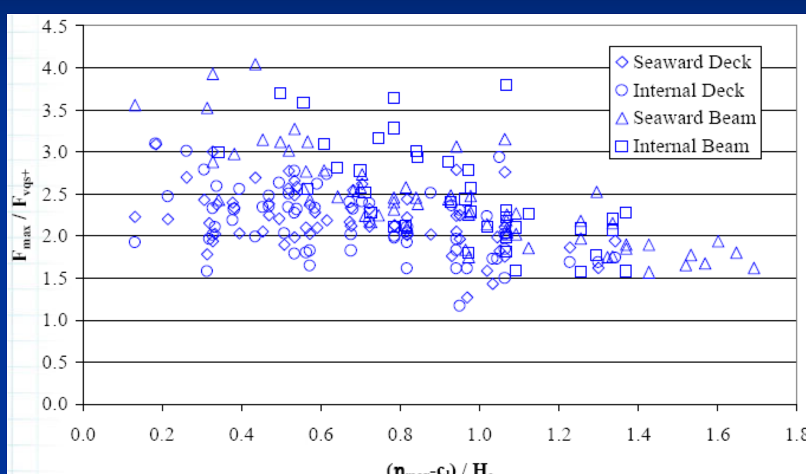
F_{qs}	quasi-static force of interest (F_{vqs+}, F_{vqs-}, F_{hqs+} or F_{hqs-})
F^*	'basic wave force', either F^*_v or F^*_h, delined in Equations (1) to (3)
c_l	clearance (distance between soffit level and still water level, SWL)
η_{max}	maximum wave crest elevation (relative to SWL)
a,b	coefficients

HR Wallingford – "Old Exponential" Method Quasi-Static

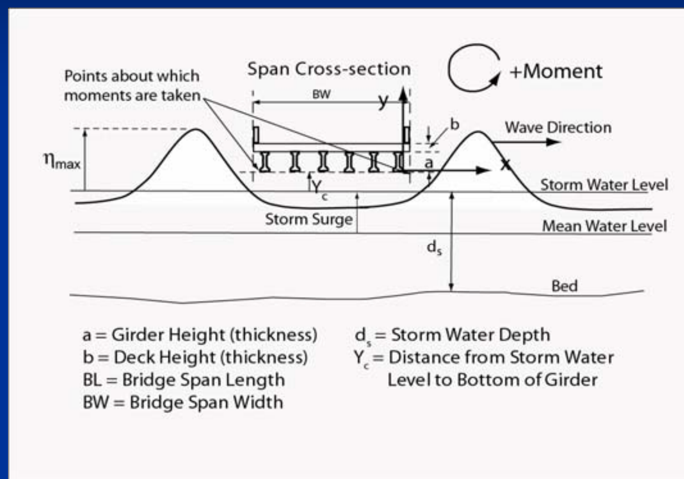
Wave load and configuration	a	b	a_{up}	b_{up}	a_{low}	b_{low}
Horizontal forces ext. & int. beam	0.72	-1.35	2.50	-1.50	0.14	-1.16
Horizontal forces external beam	0.50	-1.52	1.21	-1.65	0.05	-1.39
Horizontal forces internal beam	0.72	-2.30	2.03	-2.62	0.22	-1.99
Uplift forces ext. int elements	0.72	-0.57	1.71	-0.68	0.08	-0.47
Uplift forces external elements	0.80	-0.44	2.02	-0.54	0.17	-0.33
Uplift forces internal elements	0.63	-0.75	1.92	-0.94	0.07	-0.55
Downward forces external elements	-0.45	-1.50	-0.77	-1.29	-0.01	-0.92
Uplift forces NP external elements	0.92	-0.48	1.48	-0.62	0.49	-0.34
Uplift forces NP internal elements	0.72	-0.94	1.50	-1.15	0.13	-0.73
Uplift forces P external elements	0.76	-0.99	1.02	-1.10	0.45	-0.90
Uplift forces P internal elements	0.40	-0.79	0.80	-1.06	0.12	-0.51
Uplift forces FD external elements	0.67	-0.28	1.21	-0.46	0.19	-0.09
Uplift forces FD internal elements	0.53	-0.26	1.20	-0.65	0.08	0.13

$$\frac{F}{F^*} = a \cdot \left(\frac{\eta_{\max} - c_l}{H_s}\right)^b$$

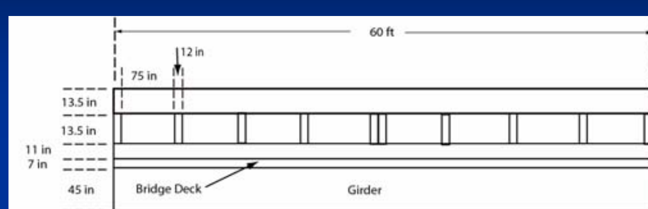
HR Wallingford – "Old Exponential" Method Impulse



Example Calculation – Escambia Bay



Example Calculation – Escambia Bay



I10 Escambia Bay Bridge Railing
(not to scale)

Example Calculation – Wave & Water Level Input

Water Density $\rho := 1.9876 \cdot \left(\frac{slug}{ft^3}\right)$

$kip := 1000 \cdot lbf$

Wave & Water Level Input

Total Water Depth	$d := 39.5 \cdot ft$
Maximum Wave Height	$Hmax := 8.2 \cdot ft$
Significant Wave Height	$Hs := 4.7 \cdot ft$
Peak Wave Period	$Tp := 4.1 \cdot sec$
Wavelength from Stream Function	$L := 92.7 \cdot ft$
Maximum Crest Height (stream function theory)	$\eta max := 4.94 \cdot ft$

Example Calculation – Structure Input

Structure Input

Clearance to Bottom of Member	$c1 := -0.3 \cdot ft + 3.75 \cdot ft$	$c1 = 3.45\, ft$
Member Depth	$bh := 2.625 \cdot ft$	
Member Width (Span Length)	$bw := 60 \cdot ft$	
Member Length (Span Width)	$bl := 35.3 \cdot ft$	

Example Calculation – "Old" Wallingford Method

Pressure at top of Member $p1 := [\eta max - (bh + c1)] \cdot \rho \cdot g$

$p1 := if(p1 < 0, 0, p1)$ $p1 = 0 \frac{lbf}{ft^2}$

Pressure at bottom of Member $p2 := (\eta max - c1) \cdot \rho \cdot g$

$p2 := if(p2 < 0, 0, p2)$ $p2 = 95 \frac{lbf}{ft^2}$

Example Calculation – "Old" Wallingford Method

Basic Vertical Wave Force $Fv := bw \cdot bl \cdot p2$

$Fv = 201812\,lbf$

Basic Horizontal Force $Fh1lat := \left[bw \cdot (\eta max - c1) \cdot \frac{p2}{2}\right]$

$Fh1lat = 4259\,lbf$

$Fh2lat := \left(bw \cdot bh \cdot \frac{p1 + p2}{2}\right)$

$Fh2lat = 7504\,lbf$

$Fhlat := if[\eta max > (c1 + bh), Fh2lat, Fh1lat]$ $Fhlat = 4259\,lbf$

Example Calculation – "Old" Wallingford Method

Upward Forces

$av1 := 0.82 \quad bv1 := 0.61 \quad Cvupper1 := 1.5 \quad cv1 := \dfrac{av1}{\left(\dfrac{\eta max - c1}{Hs}\right)^{bv1}} \quad cv1 = 1.65$

$Fvqs1 := \dfrac{Fv \cdot av1}{\left(\dfrac{\eta max - c1}{Hs}\right)^{bv1}} \quad Fvqs1 = 334\,kip \quad Fvqsu1 := Fvqs1 \cdot Cvupper1 \quad Fvqsu1 = 500\,kip$

Downward Forces

$av2 := -0.54 \quad bv2 := 0.91 \quad Cvupper2 := 1.6 \quad cv2 := \dfrac{av2}{\left(\dfrac{\eta max - c1}{Hs}\right)^{bv2}} \quad cv2 = -1.536$

$Fvqs2 := \dfrac{Fv \cdot av2}{\left(\dfrac{\eta max - c1}{Hs}\right)^{bv2}} \quad Fvqs2 = -309992\,lbf \quad Fvqsu2 := Fvqs2 \cdot Cvupper2$

$Fvqsu2 = -495987\,lbf$

Example Calculation – "Old" Wallingford Method

Shoreward Forces

$ah1 := 0.45 \quad bh1 := 1.56 \quad Chupper1 := 2 \quad ch1 := \dfrac{ah1}{\left(\dfrac{\eta max - c1}{Hs}\right)^{bh1}} \quad ch1 = 2.7$

$Fhqs1lat := \dfrac{Fhlat \cdot ah1}{\left(\dfrac{\eta max - c1}{Hs}\right)^{bh1}} \quad Fhqs1lat = 12\,kip \quad Fhqsu1lat := Fhqs1lat \cdot Chupper1$

$Fhqsu1lat = 23\,kip$

Seaward Forces

$ah2 := -0.20 \quad bh2 := 1.09 \quad Chupper2 := 2 \quad ch2 := \dfrac{ah1}{\left(\dfrac{\eta max - c1}{Hs}\right)^{bh1}} \quad ch2 = 2.701$

$Fhqs2lat := \dfrac{Fhlat \cdot ah2}{\left(\dfrac{\eta max - c1}{Hs}\right)^{bh2}} \quad Fhqs2lat = -3\,kip$

$Fhqsu2lat := Fhqs2lat \cdot Chupper2 \quad Fhqsu2lat = -6\,kip$

Example Calculation – "Old" Wallingford Method

Vertical Impact Load	$m := 2.3$	$Fvmax := Fvqs1 \cdot m$	$Fvmax = 767\ kip$
Horizontal Impact Load	$m := 3.6$	$Fhlatmax := Fhqs1lat \cdot m$	$Fhlatmax = 41\ kip$

Scott Douglass Method

- Basically same as "Old Exponential" Wallingford Method
- Vertical & Horizontal Quasi-Static Coefficient
  - Recommends 1.0 with Factor of Safety of 2.0
- Horizontal – Multiple Beams - Recommends using 40% of External Beam Load
- Vertical Impact Coef - Recommends 4.0
- Horizontal Impact Coef - Recommends 7.0

Scott Douglass Method

Vertical Quasi-Static Forces

Upward Forces

$av1 := 1$ $cv1 := av1$ $Cvupper1 := 2.0$

$Fvqs1 := Fv \cdot cv1$ $Fvqs1 = 202\ kip$

$Fvqsu1 := Fvqs1 \cdot Cvupper1$ $Fvqsu1 = 404\ kip$

Horizontal Quasi-Static Forces

Shoreward Forces

$ah1 := 1.0$ $ch1 := ah1$ $Chupper1 := 2$

$Fhqs1lat := Fhlat \cdot ch1$ $Fhqs1lat = 4\ kip$

$Fhqsu1lat := Fhqs1lat \cdot Chupper1$ $Fhqsu1lat = 9\ kip$

Scott Douglass Method

Wave Impact Forces

Vertical Impact Load

$cvi := 3$

$Fvmax := Fvqs1 + cvi \cdot Fv$ $Fvmax = 807\ kip$

Horizontal Impact Load

$chi := 6$

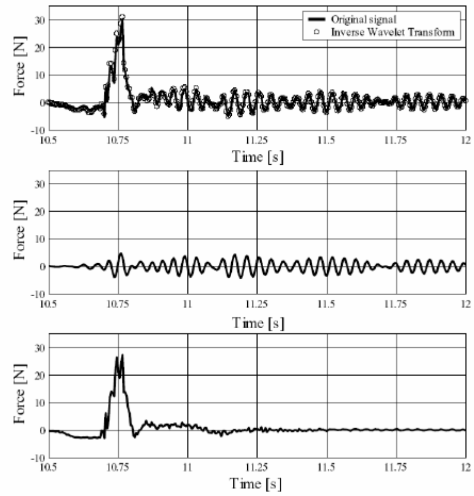
$Fhlatmax := Fhqs1lat + chi \cdot Fhlat$ $Fhlatmax = 30\ kip$

HR Wallingford – "New Linear" Method

Original signal

Response of measurement instrument

Cleaned signal



Cuomo (2005)
Cuomo et al (2003)

HR Wallingford – "New Linear" Method

Dimensionless wave forces:

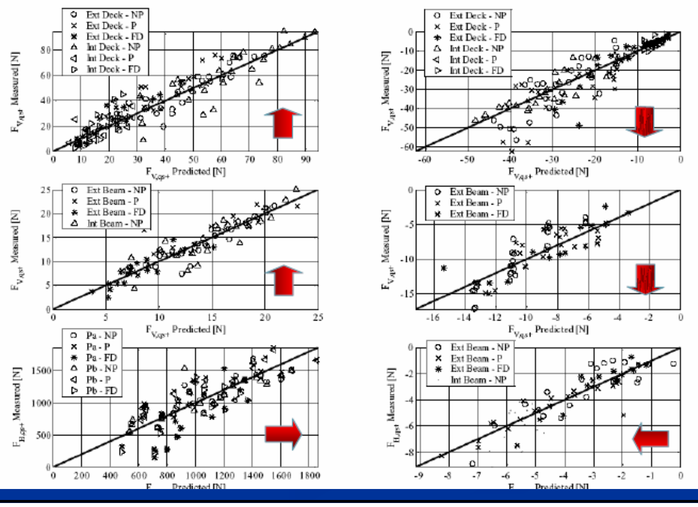
- Quasi-static (F^*_{qs}): $F^*_{qs+} = \dfrac{F_{qs+,1/250}}{\rho g H_S \cdot A}$ $\quad F^*_{qs-} = \dfrac{F_{qs-,1/250}}{\rho g H_S \cdot A}$
- Impulsive (F^*_{imp}): $F^*_{imp} = \dfrac{F_{imp,1/250}}{\rho g H_S \cdot A}$

Dimensionless hydrostatic head: $\eta^* = \dfrac{\eta_{\max} - c_l}{d}$

Cuomo (2005)
Allsop et al. (2006)

HR Wallingford – "New Linear" Method

$$\eta^* = \frac{\eta_{max} - c_l}{d} \Rightarrow F^*_{qs} = a \cdot \eta^* + b \Rightarrow F_{qs} = F^*_{qs} \cdot (\rho g H_S A)$$



H_s = significant wave height

η = crest elevation

c_l = clearance

d = water depth

A = exposed area

ρ = water density

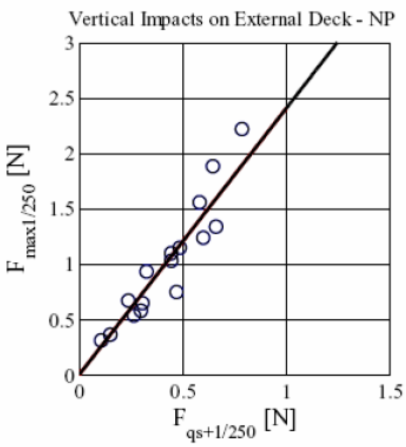
a, b = empirical coefficients

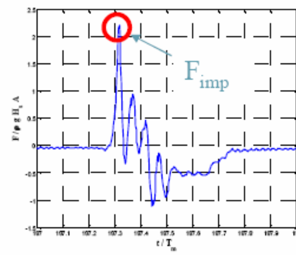
HR Wallingford – "New Linear" Method Quasi Static

Parameter	Direction	Element	Position	Config.	a	b	R^2	s_ε
Pressure	Horizontal	Beam Pa & Pb	Ext	FD	1.19	0.43	0.90	0.34
Pressure	Horizontal	Beam Pa & Pb	Ext	P	1.19	0.43	0.87	0.22
Pressure	Horizontal	Beam Pa & Pb	Ext	NP	1.19	0.43	0.96	0.17
Force	Horizontal	Beam	Int	NP	0.56	0.75	0.90	6.84
Force	Vertical	Beam	Ext	FD	1.74	0.14	0.96	1.68
Force	Vertical	Beam	Ext	P	0.71	0.57	0.97	1.24
Force	Vertical	Beam	Ext	NP	1.10	0.46	0.96	1.61
Force	Vertical	Beam	Int	NP	1.36	0.46	0.89	2.27
Force	Vertical	Deck	Ext	FD	2.31	0.05	0.95	6.78
Force	Vertical	Deck	Ext	P	1.23	0.51	0.96	7.28
Force	Vertical	Deck	Ext	NP	1.57	0.52	0.84	7.64
Force	Vertical	Deck	Int	FD	0.83	0.13	0.69	9.80
Force	Vertical	Deck	Int	P	0.58	0.19	0.67	6.57
Force	Vertical	Deck	Int	NP	1.57	0.73	0.95	11.21

Table 1 - coefficients a and b for fit lines and values of R^2 for equation 18 and 19, positive loads; s_ε in model units: pressure [kPa] and force [N]

HR Wallingford – "New Linear" Method Impulse





$$F_{imp} = a \cdot F_{qs+} + b$$

HR Wallingford – "New Linear" Method Impulse

Parameter	Direction	Element	Position	Config.	a	R^2	s_ε
Force	Horizontal	Beam	Ext	All	2.45	0.90	1.10
Force	Horizontal	Beam	Int	NP	3.35	0.89	25.62
Force	Vertical	Beam	Ext	FD	2.87	0.94	5.38
Force	Vertical	Beam	Ext	P	1.74	0.48	1.24
Force	Vertical	Beam	Ext	NP	2.28	0.32	3.41
Force	Vertical	Deck	Ext	FD	2.35	0.93	15.81
Force	Vertical	Deck	Ext	P	1.99	0.64	7.28
Force	Vertical	Deck	Ext	NP	2.22	0.85	20.41
Force	Vertical	Beam	Int	NP	2.59	0.69	8.36
Force	Vertical	Deck	Int	FD	2.35	0.98	21.32
Force	Vertical	Deck	Int	P	1.84	0.88	6.57
Force	Vertical	Deck	Int	NP	2.29	0.96	26.60

Table 3 - coefficients a for fit lines and values of R^2 for equation 22; s_ε in model units: pressure [kPa] and force [N]

Wave Forces
Modified Kaplan Method

Wave Forces on Bridge Spans

- Composed of several components
  - Drag
  - Inertia
  - Change in added mass
  - Buoyancy (vertical only)
  - Slamming

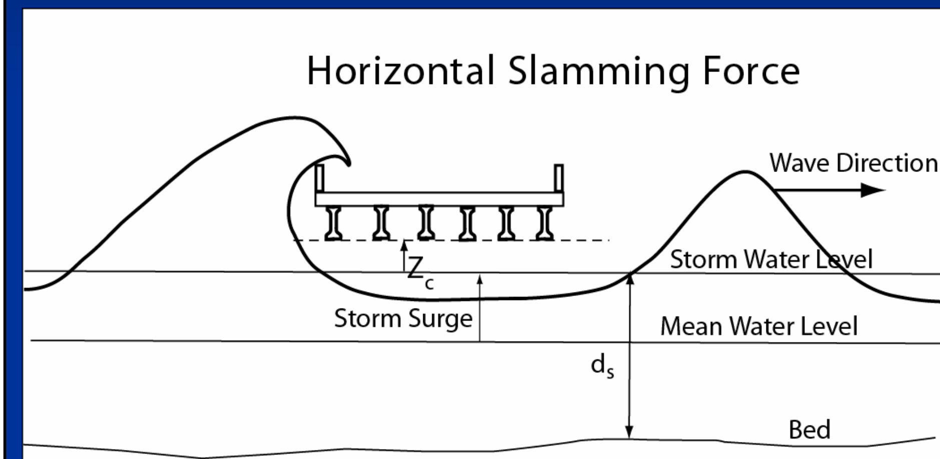
Wave Forces on Bridge Spans (cont.)

- First four components can be addressed directly with the Modified Kaplan Method
  - Modified Kaplan
    - Drag
    - Inertia
    - Change in added mass
    - Buoyancy (vertical only)

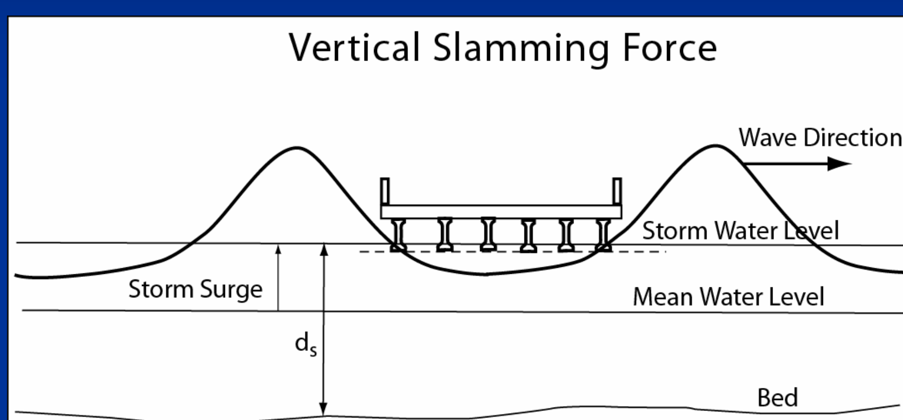
Wave Forces on Bridge Spans (cont.)

- The slamming force occurs when the air-water interface strikes the structure
  - Horizontal slamming force (breaking waves)
  - Vertical slamming force (when low member elevation is above wave trough elevation and below wave crest elevation)

Wave Forces on Bridge Spans (cont.)



Wave Forces on Bridge Spans (cont.)

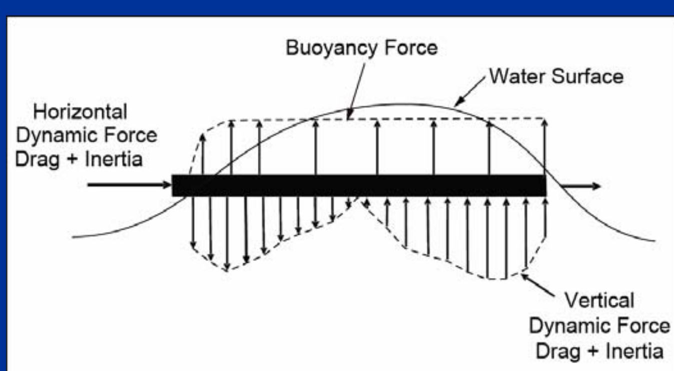


Modified/Extended Kaplan Method

- Developed for Bridge Super Structure Shapes
  - Girders – possible air entrapment
- Shorter waves T ~ 4 - 8 sec (wave lengths 80 ft to 225 ft)
  - Larger velocity and acceleration gradients
  - Larger buoyancy force gradients
  - Larger change in added mass components

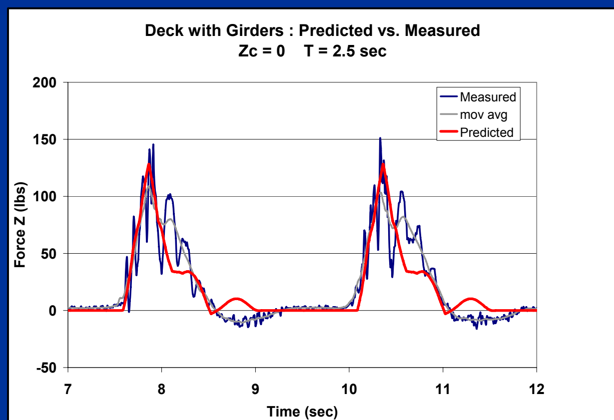
Modified/Extended Kaplan Method

- Moments as well as forces essential to computing structural response



Modified/Extended Kaplan Method

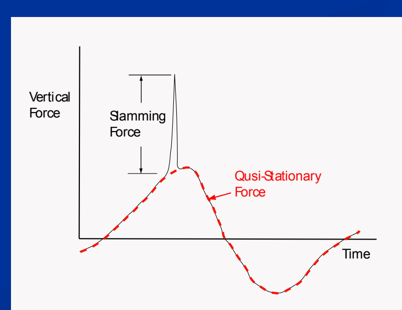
- Vertical Force
  - quasi-stationary force
  - Slamming force



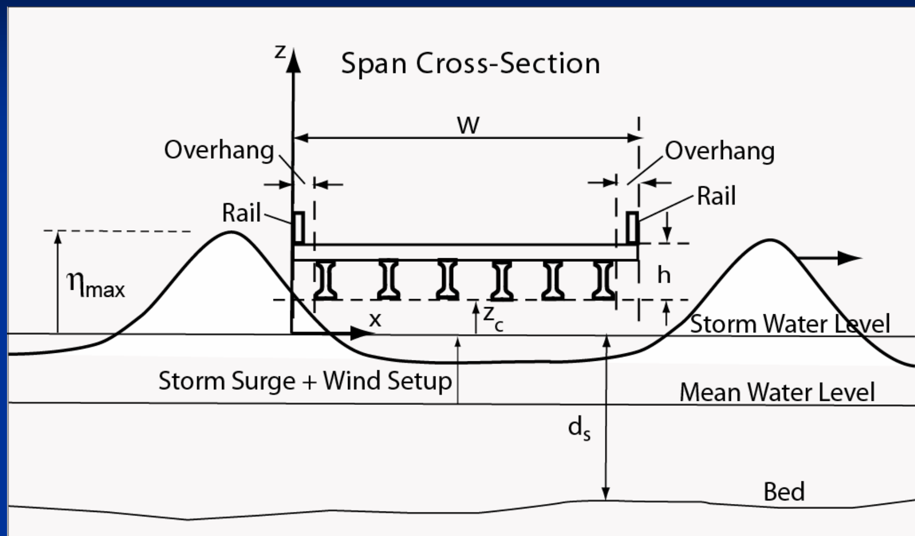
Modified/Extended Kaplan Method

- Slamming Force
  - Magnitude?
  - Duration?
  - Spatial distribution?
- These Questions Must Be Answered

Before its Impact on Structural Response Can Be Determined



Definition Sketch



Modified/Extended Kaplan Method

quasi-Static Force

$$F_{Horizontal} \equiv F_x = F_{Drag} + F_{Inertia} + F_{CAM}$$

$$F_{Vertical} \equiv F_z = F_{Buoyancy} + F_{Drag} + F_{Inertia} + F_{CAM}$$

Modified/Extended Kaplan Method

quasi-Static <u>Vertical</u> Force

$$F_z = \frac{d\left(m_{a(z)} V_z\right)}{dt} + F_{drag} + F_{buoyancy}$$

$$= \frac{d\left(m_{a(z)} V_z\right)}{dt} + \frac{1}{2}\rho\, L\, w\, C_{d(z)}\, V_z \left|V_z\right| + F_{buoyancy}$$

$$\frac{d\left(m_{a(z)} V_z\right)}{dt} = \frac{dm_{a(z)}}{dt} V_z + m_{a(z)} \frac{dV_z}{dt}$$

Modified/Extended Kaplan Method

quasi-Static <u>Vertical</u> Force (cont.)

$$m_{a(z)} \equiv \text{added mass} = \frac{C_{m(z)}\, \pi\, \rho\, L\, w(t)^2}{4\sqrt{1+\left(\frac{w(t)}{L}\right)^2}} \left(C_1 + C_2 \frac{h(t)}{L} + C_3 \sqrt{\frac{h(t)}{w(t)}}\right)$$

$\rho \equiv$ Density of Water

$w \equiv$ Wetted Span Width

$L \equiv$ Span Length

$h \equiv$ Wetted Span Height

$t \equiv$ Time

Modified/Extended Kaplan Method

quasi-Static Vertical Force (cont.)

- Buoyancy Force

$$F_b = \rho\, g\, L \iint_{wcsa} dA$$

F_b = Buoyancy force

wcsa $\equiv$ wetted cross-sectional area

Modified/Extended Kaplan Method

quasi-Static Vertical Force (cont.)

$$F_z(t) = \int_{t=0}^{t} \left\{ \begin{aligned} &\iint_{wcsa} \left[\frac{d\left(m_{a(z)} V_z\right)}{dt} + F_{buoyancy} \right] dA + \iint_{pa} F_{drag}\, dA \\ &= \iint_{wcsa} \left(\frac{dm_{a(z)}}{dt} V_z + m_{a(z)} \frac{dV_z}{dt} + \rho\, g\, L \right) dA + \iint_{pa} \left(\frac{1}{2} \rho\, L\, w(t)\, C_{d(z)}\, V_z \left|V_z\right| \right) dA \end{aligned} \right\} dt$$

wcsa $\equiv$ wetted cross-sectional area

pa $\equiv$ projected area

Modified/Extended Kaplan Method

quasi-static Horizontal Force

$$F_x = \frac{d\left(m_{a(x)} V_x\right)}{dt} + F_{drag}$$

$$= \frac{d\left(m_{a(x)} V_x\right)}{dt} + \frac{1}{2} \rho \; L \; h \; C_{d(x)} \; V_x \; \left|V_x\right|$$

$$\frac{d\left(m_{a(x)} V_x\right)}{dt} = \frac{dm_{a(x)}}{dt} V_x \; + \; m_{a(x)} \frac{dV_x}{dt}$$

Modified/Extended Kaplan Method

quasi-static Horizontal Force (cont.)

$$m_{a(x)} \equiv \text{ added mass } = \frac{C_{m(x)} \; \pi \; \rho \; L \; h(t)^2}{4\sqrt{1+\left(\frac{h(t)}{L}\right)^2}} \left(C_1 + C_2 \frac{w(t)}{L} + C_3 \sqrt{\frac{w(t)}{h(t)}} \right)$$

$\rho \equiv$ Density of Water

$w \equiv$ Wetted Span Width

$L \equiv$ Span Length

$h \equiv$ Wetted Span Height

$t \equiv$ Time

Modified/Extended Kaplan Method

quasi-static Horizontal Force (cont.)

$$F_x(t) = \int_{t=0}^{t} \left\{ \begin{aligned} &\iint_{wcsa} \left[\frac{d\left(m_{a(x)} V_x\right)}{dt} + F_{buoyancy} \right] dA + \iint_{pa} F_{drag}\, dA \\ &= \iint_{wcsa} \left(\frac{dm_{a(z)}}{dt} V_z + m_{a(x)} \frac{dV_x}{dt} \right) dA + \iint_{pa} \left(\frac{1}{2} \rho\, L\, w(t)\, C_{d(z)}\, V_x \left|V_x\right| \right) dA \end{aligned} \right\} dt$$

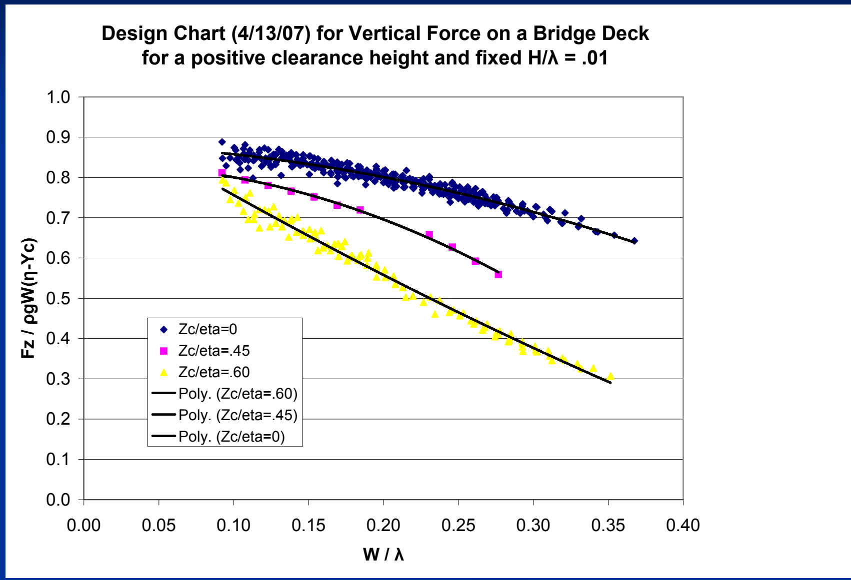
wcsa ≡ wetted cross-sectional area

pa ≡ projected area

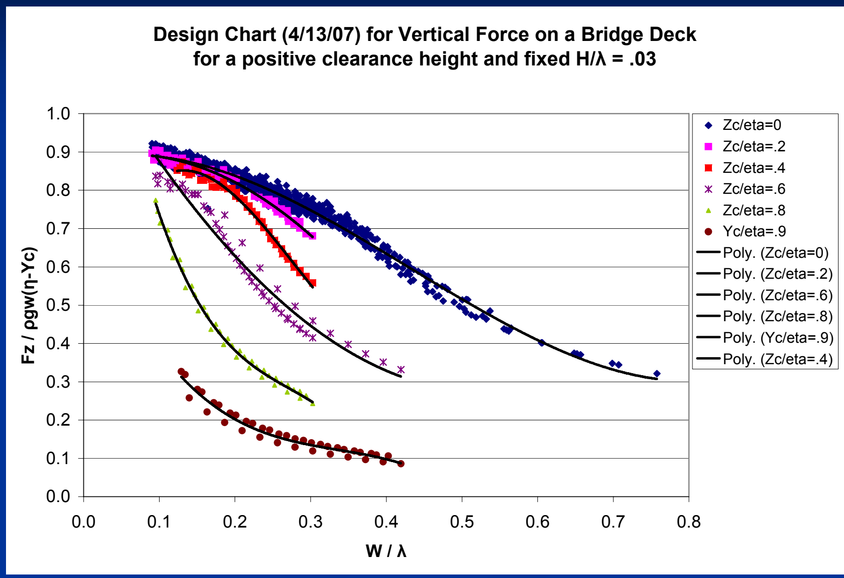
Design Wave Force Plots

- Procedure for development of plots
  - Test equations with laboratory and field data
  - Use equations to generate simulations for wide range of conditions
  - Use data to evaluate dimensionless groups
  - Dimensionless groups
    - Fx, Fz and M in terms of known water, wave, and structure variables

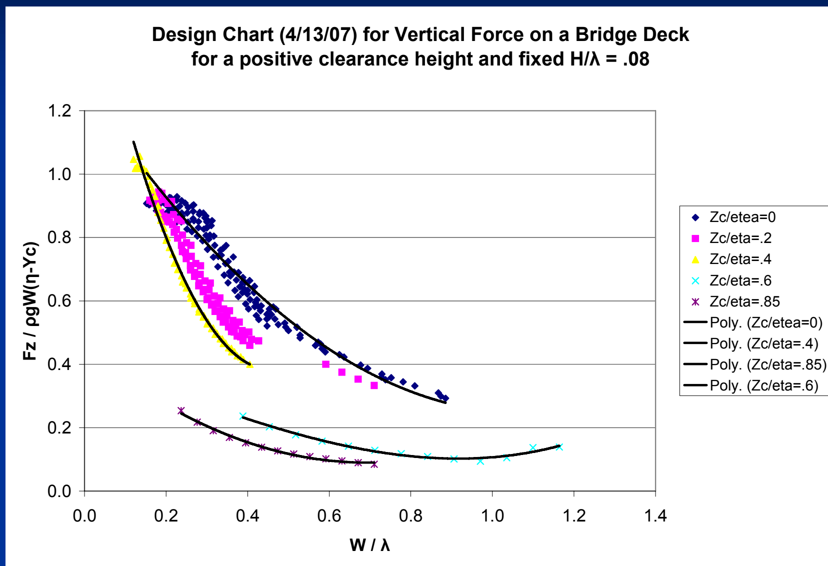
Sample Wave Force Plots



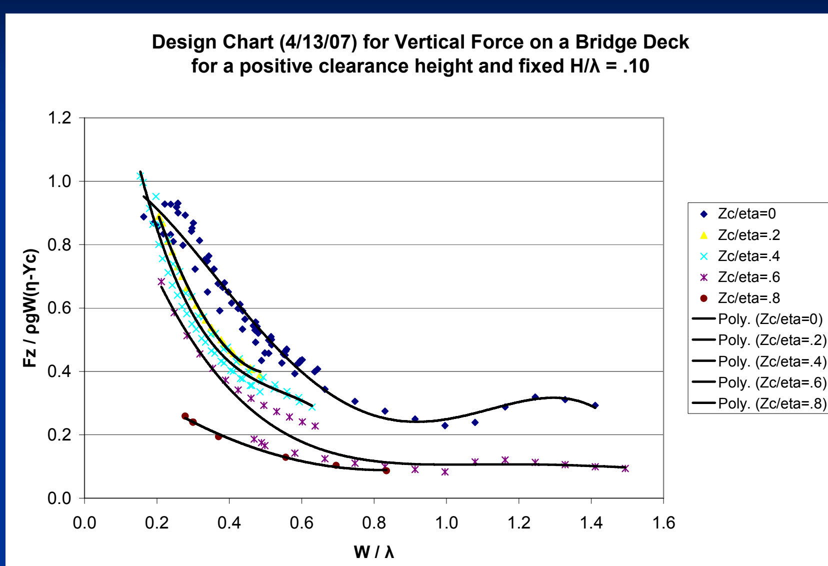
Sample Wave Force Plots



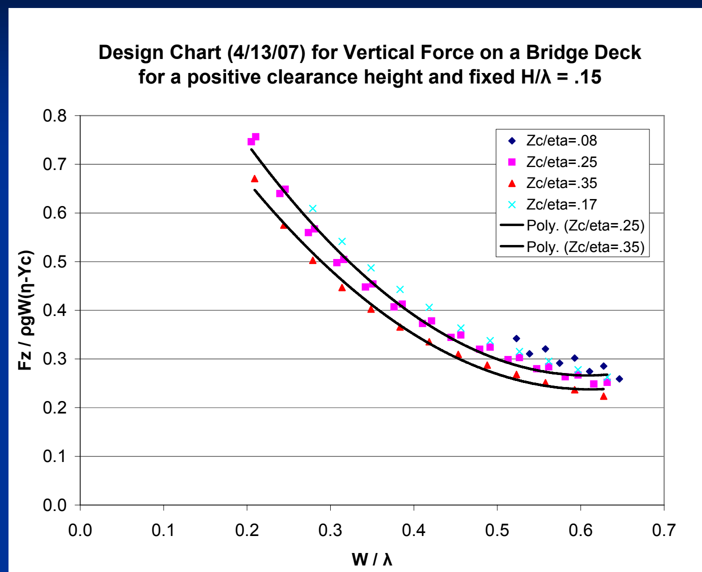
Sample Wave Force Plots



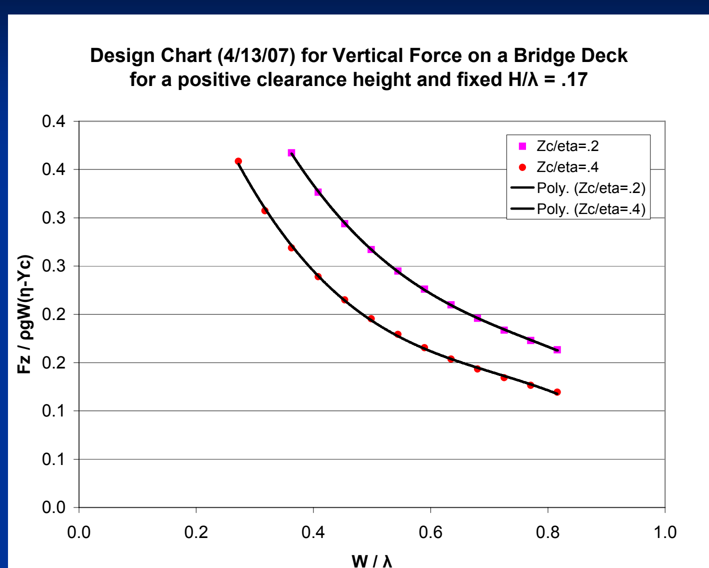
Sample Wave Force Plots



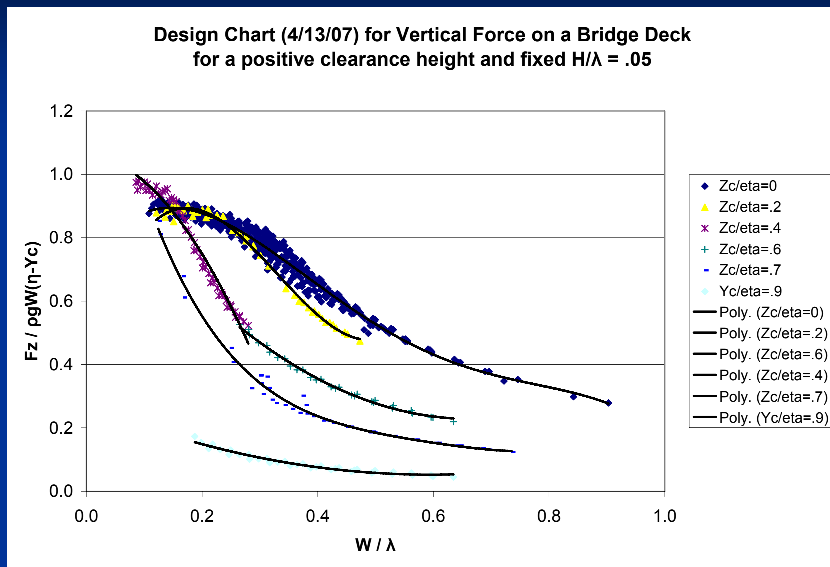
Sample Wave Force Plots



Sample Wave Force Plots



Sample Wave Force Plots



Outline (cont.)

- Comparisons Between Predicted and Measured ((Max & Jeff)
  - Laboratory data
  - Field data
  - Parametric study

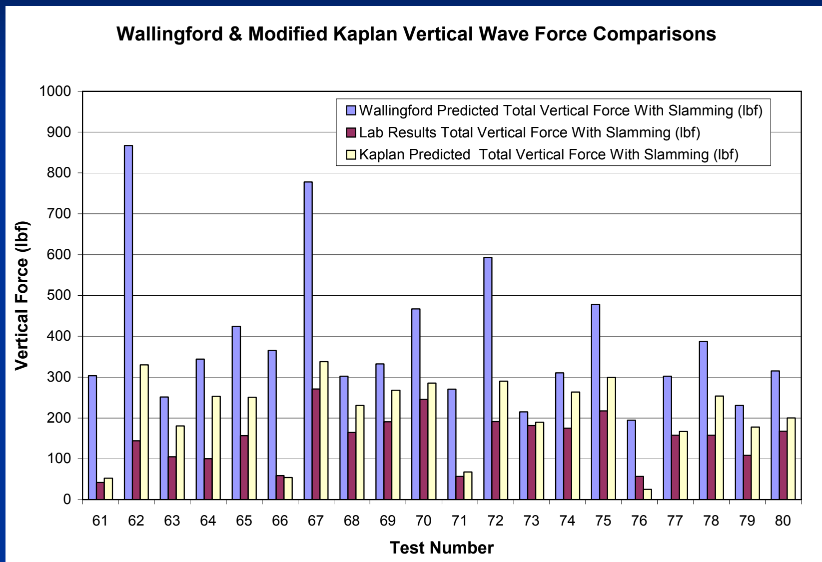
Comparison of Wave Force Methods

Wallingford
Modified Kaplan

Comparisons with UF Laboratory Data

- The maximum horizontal and vertical wave forces were computed using the three methods and the results compared with the measured values

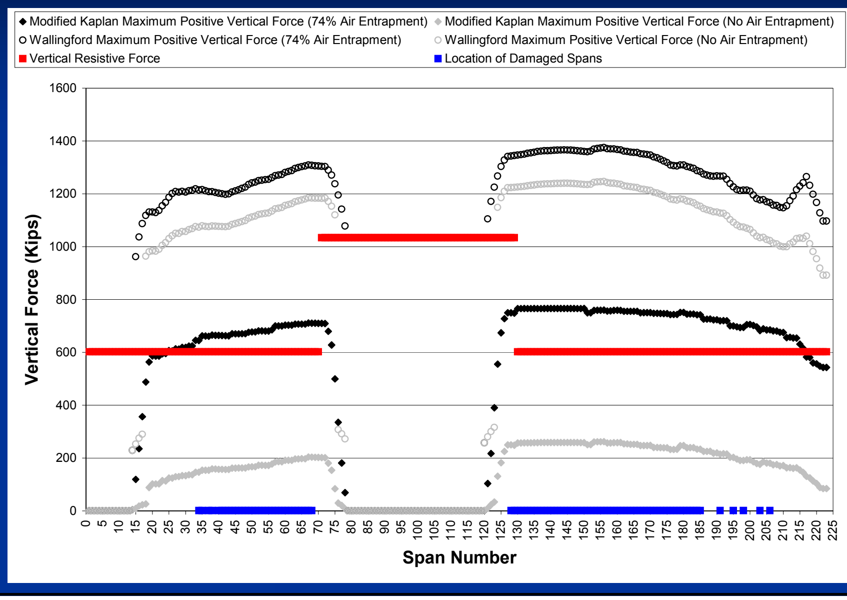
Comparisons with UF Laboratory Data (cont.)



Comparisons with I-10 Escambia Bay Bridge Data

- The maximum horizontal and vertical wave forces were computed using the three methods for the water, wave and structure parameters for the I-10 Escambia Bay Bridge. The results were compared with computed resistive forces (weight + tie-down) and measured damage.

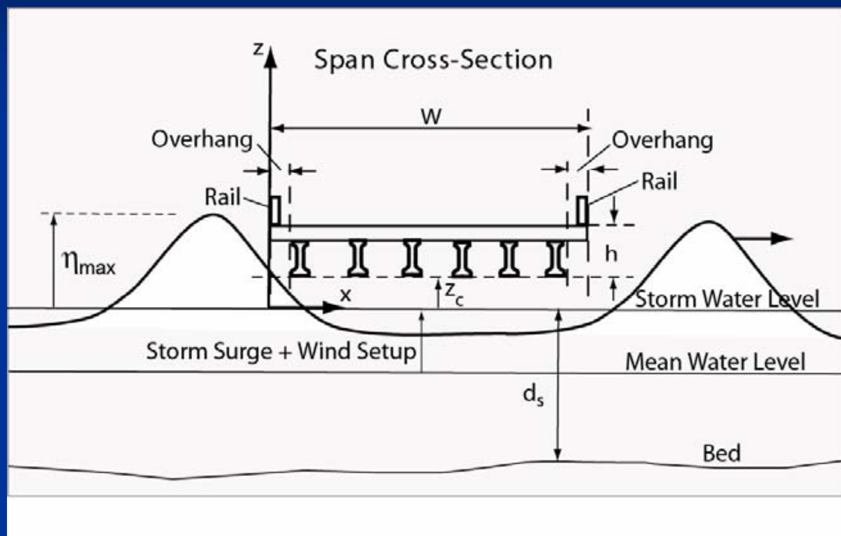
Comparisons with I-10 Escambia Bay Bridge Data (cont.)



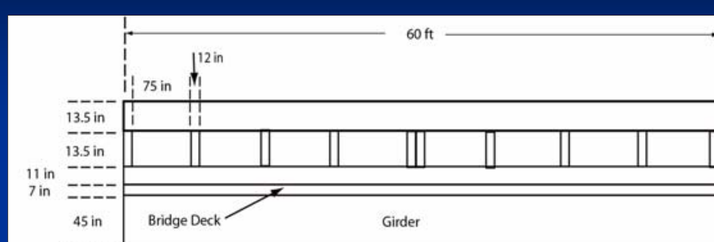
Comparisons for Hypothetical Water, Wave and I10 – Escambia Bay Spans

- The maximum horizontal and vertical wave forces were computed using the three methods for hypothetical water, wave and bridge spans. In this case there are no measured values for comparison.

Comparisons for Hypothetical Water, Wave and I10 – Escambia Bay Spans



Comparisons for Hypothetical Water, Wave and I10 – Escambia Bay Spans

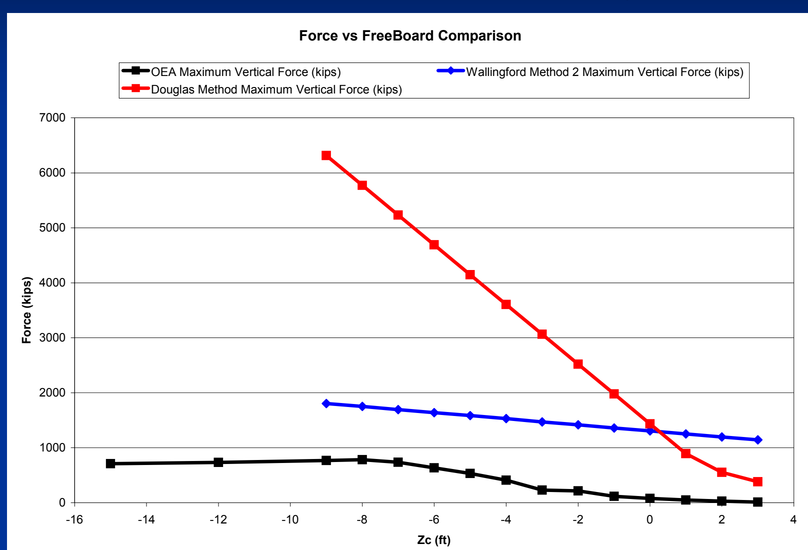


I10 Escambia Bay Bridge Railing
(not to scale)

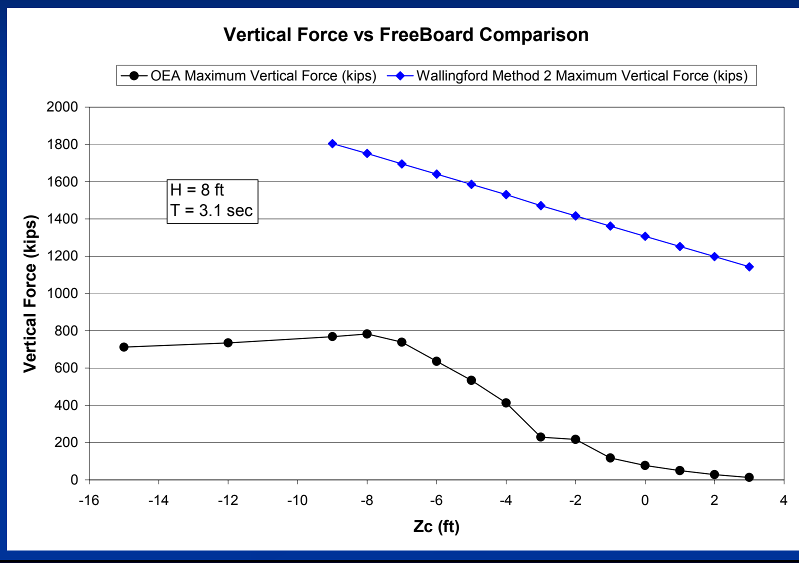
Comparisons for Hypothetical Water, Wave and I10 – Escambia Bay Spans

Wave Period (sec)	Maximum Wave Height (ft)	Z_c (ft)
3.1	8	3
3.1	8	2
3.1	8	1
3.1	8	0
3.1	8	-1
3.1	8	-2
3.1	8	-3
3.1	8	-4
3.1	8	-5
3.1	8	-6
3.1	8	-7
3.1	8	-8
3.1	8	-9

Comparisons for Hypothetical Water, Wave and I10 – Escambia Bay Spans



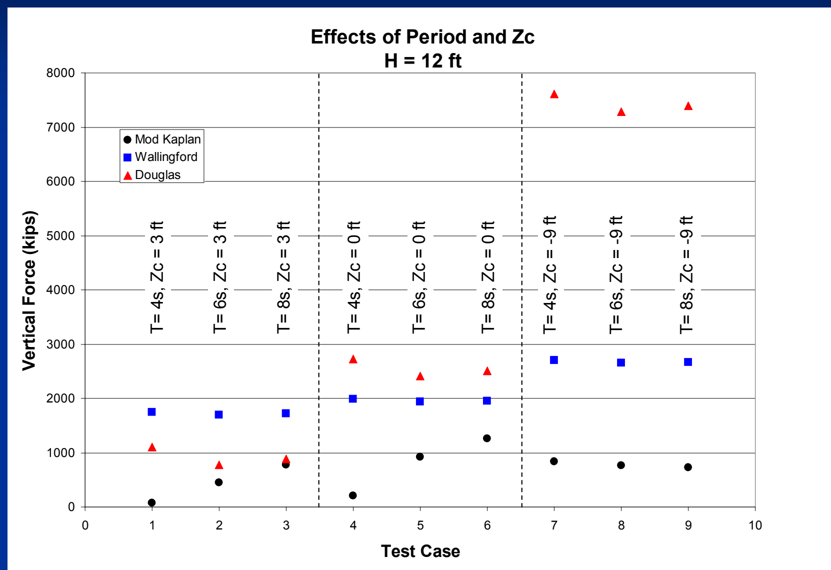
Comparisons for Hypothetical Water, Wave and I10 – Escambia Bay Spans



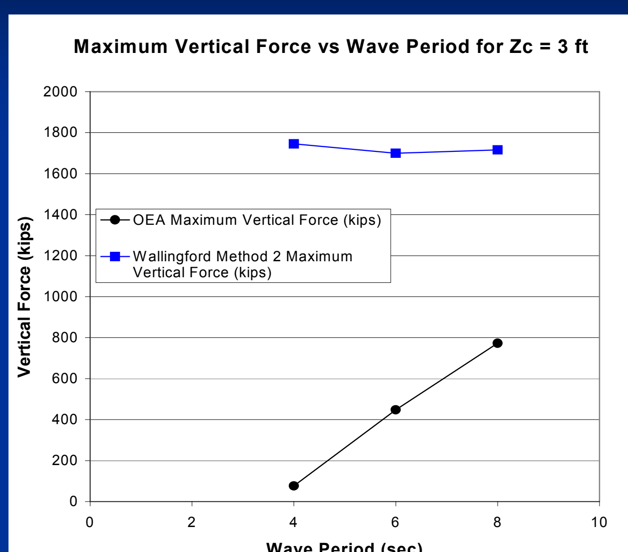
Comparisons for Hypothetical Water, Wave and I10 – Escambia Bay Spans

Wave Period (sec)	Maximum Wave Height (ft)	Z_c (ft)
4	12	3
6	12	3
8	12	3
4	12	0
6	12	0
8	12	0
4	12	-9
6	12	-9
8	12	-9

Comparisons for Hypothetical Water, Wave and I10 – Escambia Bay Spans



Comparisons for Hypothetical Water, Wave and I10 – Escambia Bay Spans



Outline

- Selection Criteria (John)
- Recommended method (John)
- Future work plans for wave force (Max)

Basis Of A Choice

- Relationship with experimental results
- Prediction of failures in field
- Theoretical Completeness
- Practicality

Relationship with Experimental Results

- Douglas et al – Just getting started
- Texas A & M? – nothing published
- Wallingford – wrong configuration and no submergence or period effect
- Mississippi State? – fixed configuration- may still be able to do reality checks
- Issacson – flat plate, no slamming– will still do reality check

Relationship with Experimental Results

- Dan Cox – just getting started in OSU’s large wave tank – have contacted Max for info on UF’s test protocols
- University of Florida – all we have for thorough investigation of bridges

Correlation With Observed Performance

- Already presented
  - Lab tests favor modified Kaplan
  - Modified Kaplan tracks Escambia Bay failures better
  - Need to review data fitness as calibrated

Theoretical Completeness

- Consistent with Physics
  - Modified Kaplan involves pressure, drag, inertia and wave period
- Adaptable for future developments
- Defensible and Satisfying

Practicality

- Given to Consistent Results
  - Few anomalies when compared to tests
  - "Test Drives" needed
- Strive to be user friendly
  - Table look up either way
- Intuitive – clear trends & components of solution identifiable
  - Wallingford's simple starting point seems attractive, but empirical coefficients hide complexity

Is a Hybrid Method Possible

- Can we combine Wallingford simple starting model with UF's data?
- Based on reassessment of Wallingford:
  - Still does not account for wave period
  - Still does not include submergence
  - Not adaptable to full range of parameters that needs to be considered
  - Line fit does not have force=0 when crest does not hit structure.

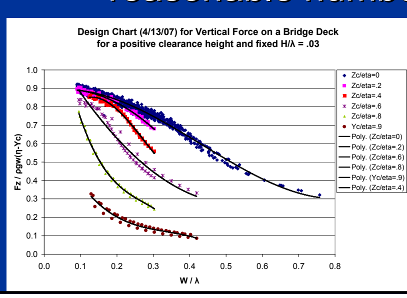
$$F^*_{qs} = a \bullet \eta^* + b \qquad \eta^* = (\eta_{max} - c_l) / d)$$

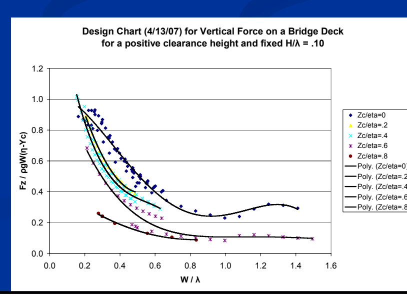
Is a Hybrid Method Possible

- Conclusion – not a promising idea

Recommendation

- Proceed with Modified Kaplan
- Use graphical/tabular presentation for forces, not basic equations.
  - Remember Max's figures, need to have a reasonable number





Further Confirming Studies

- Comparison to Wallingford data
- Refinements based on forthcoming UF data with railings, wider beam spacing, etc
- Comparisons with work of others just getting underway

Outline (cont.)

- Wave Force input Parameters (Max & Jeff)
  - Design water elevation - spec
  - Design wave parameters - spec
- Step through tentative specification
- Load Factor Modifiers Based On Met/Ocean Joint Probability
  - Information from four hurricane hindcasts
  - Information from hurricane path statistics
  - Method for obtaining modifiers
  - Only applicable to Level I and II analyses

Excerpts from the Draft Specifications

- Work In-Progress

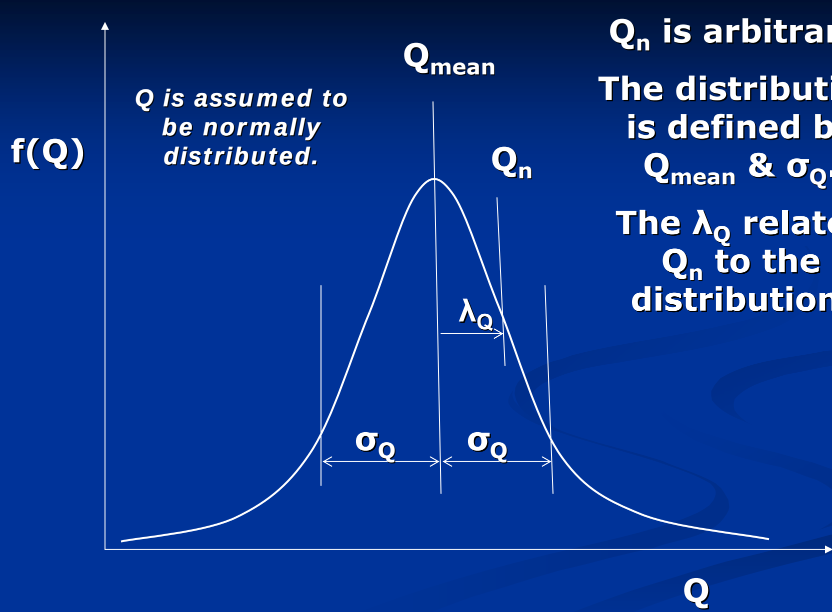
Code Calibration

Selection of load and resistance factors (γ's & φ's) to achieve a target level of reliability (in other words, probability of failure)

Load Factors

Table 3.4.1-1 – Load Combinations & Load Factors

LOAD FACTORS / LOAD COMBINATIONS	DC DD DW EH EV ES EL	LL IM CE BR PL LS	WA	WS	WL	FR	TU CR SH	TG	SE	EQ	one at a time IC	CT	CV
strength I	γ_P	1.75	1.00	-	-	1.00	0.50 / 1.20	γ_{TG}	γ_{TG}	-	-	-	-
strength II	γ_P	1.35	1.00	-	-	1.00	0.50 / 1.20	γ_{TG}	γ_{TG}	-	-	-	-
strength III	γ_P	-	1.00	1.40	-	1.00	0.50 / 1.20	γ_{TG}	γ_{TG}	-	-	-	-
strength IV EH, EV, ES, DW DC only	γ_P 1.5	-	1.00	-	-	1.00	0.50 / 1.20	-	-	-	-	-	-
strength V	γ_P	1.35	1.00	0.40	1.00	1.00		γ_{TG}	γ_{TG}	-	-	-	-
extreme-event I	γ_P	γ_Q	1.00	-	-	1.00	-	-	-	1.00	-	-	-
extreme-event II	γ_P	0.50	1.00	-	-	1.00	-	-	-	-	1.00	1.00	1.00
service I	1.00	1.00	1.00	0.30	1.00	1.00	1.00 / 1.20	γ_{TG}	γ_{TG}	-	-	-	-
service II	1.00	1.30	1.00	-	-	1.00	1.00 / 1.20	-	-	-	-	-	-
service III	1.00	0.80	1.00	-	-	1.00	1.00 / 1.20	γ_{TG}	γ_{TG}	-	-	-	-
service IV	1.00	-	1.00	0.70	-	1.00	1.00 / 1.20	-	1.0	-	-	-	-
fatigue-and-fracture	-	0.75	-	-	-	-	-	-	-	-	-	-	-



Q_n is arbitrary.

The distribution is defined by Q_{mean} & σ_Q.

The λ_Q relates Q_n to the distribution.

Data Required for Calibration

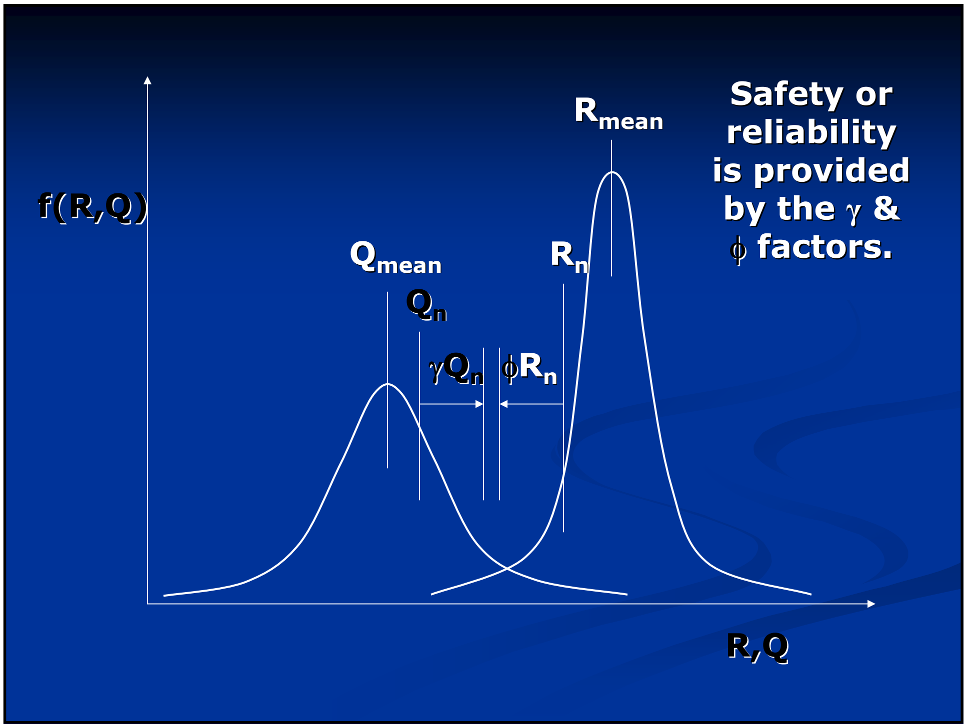
The mean of the load distribution, Q_{mean}

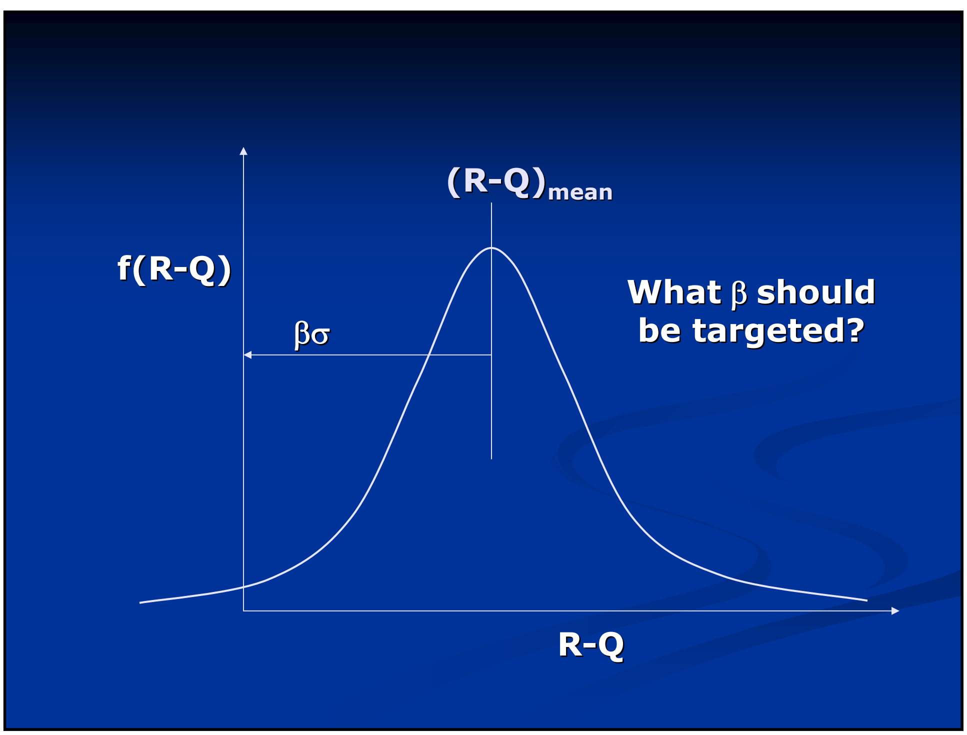
The standard deviation of the load distribution, σ_Q ($COV=\sigma_Q/Q_{mean}$)

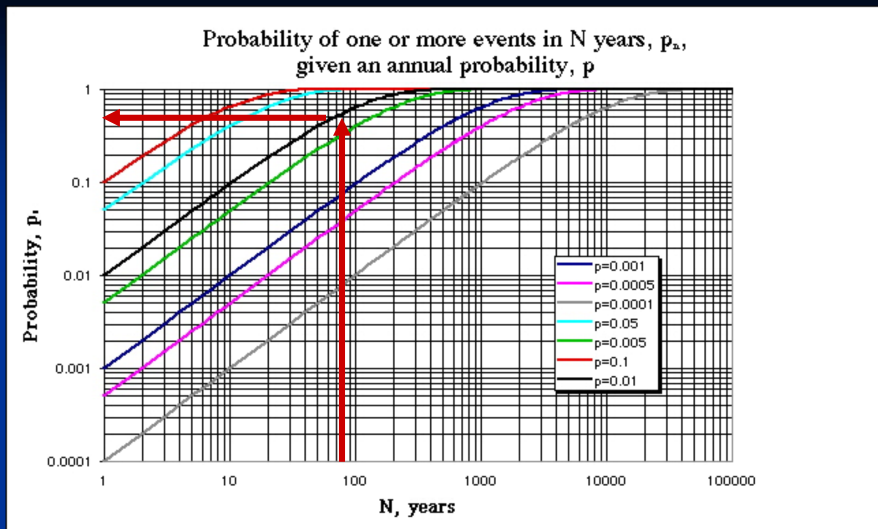
The bias of the load distribution, λ_Q

Choice of the Nominal Load, Q_n

The nominal load, Q_n, can be arbitrarily chosen as long as no unfactored service limit state is specified. No service limit state is being considered for coastal-storm force effects.


Safety or reliability is provided by the γ & ϕ factors.
R_mean
f(R,Q)
Q_mean
R_n
Q_n
γQ_n
ϕR_n
R,Q


(R-Q)_mean
f(R-Q)
What β should be targeted?
βσ
R-Q



$$p_n = 1 - (1 - p)^N$$

Annual v. 75-year Probability of Occurrence

Annual probability of occurrence of 0.01 is basically equivalent to a 75-year probability of occurrence of 0.5 (in other words, the bias, λ_Q, is about 1.0) .

Monte-Carlo Simulation

Monte-Carlo simulation is used to determine the load factor for coastal-storm force effects by trial-and-error using assumed load combinations and the existing load factors for non-coastal-storm force effects & existing resistance factors.

Outline of Procedure

- Loop with trial load factors
  - Loop for sample bridges
    - Loop for Eta-max (COV=0.4, 0.3??)
      - Loop for Zc (COV=0.5, 0.3??)
        - Loop for wave length (COV=0.4, 0.3??)
        - Monte Carlo for forces (with COV's)
        - Continue
      - Continue
    - Continue
  - Continue – have force distribution for each combination of eta, Zc and wavelength for a given bridge
  - Monte Carlo for Betas
- Continue

Execute for Level I and III

Load Factor Modifiers

- Possibly based on wedge of wind attack angles for wave and wind setup compared to circle or reduced circle, e.g. 300 degrees
- May be function of Zc, e.g. 1.0 for Zc<0

Schedule

- 50% deliverables – mid May, June Meeting
- 90% Spec & Manual - 7/31
- Base Load factors
- Load Factor Modifiers
- UF continuing work
- Task 5

Outline

- Status of FLDOT work
  - Future plans and schedule
  - Potential impact to this project
  - Reassessment of retrofit

Outline (cont.)

- Revised Bridge Screening Methodology (Max)
  - Issues addressed
    - Factors affecting design water levels
    - Factors affecting design wave parameters
    - Span elevation
    - Span type
    - Potential for air entrapment
    - Importance of structure

Florida Projects

- Pilot Study – Phase I
  - Screening Criteria (Completed)
  - Screening (Completed)
- Pilot Study – Phase II
  - Refinement of Met/Ocean Parameters (Completed)
  - Computation of Wave Forces (Completed)
  - Refinement of Screening Criteria (In Progress)

Florida Projects (cont.)

- Met/Ocean Joint Probability Study
  - Analysis of 4 Hurricane Hindcasts (Completed)
    - Ivan
    - Katrina
    - Francis
    - Jeanne
- Joint Probability Approach
  - Load Factor Modifiers (Proposed)
    - Use 100 year worst case met/ocean conditions
    - Develop load factor modifiers that depend on site specific conditions

Florida Projects (cont.)

- Laboratory Wave Tank Tests
  - Flat Deck Tests (completed)
  - Deck with Girders
    - 230 Tests Completed
    - Data Analysis (In Progress)
  - Deck with Girders and Overhangs
    - Ready to Install
    - Limited Tests

Florida Projects (cont.)

  - Deck with Girders and Overhangs and Rails
    - Ready to Install
    - Complete Range of Tests
- Deck with Girders and Overhangs and Rails
  - Wider Girder Spacing
  - Limited Tests
- Slamming Force Tests
  - Flat Deck
  - Instrumentation
    - Pressure Mats and Loadcells

Schedule

TASK	Date shown in Work Plan	PROPOSED COMPLETION DATES
Notice to Proceed	September 1, 2006	
Kickoff Meeting	December 4,5,6, 2006	
Task 2	December 15, 2006	January 15, 2007
Task 3	December 15, 2006	February 28, 2007
Task 4	January 26, 2007	March 31, 2007
Task 5	March 2, 2007	April 30, 2007
Task 6 50% Draft Specification and Manual 90% Draft Specification and Manual 100% Draft Specification and Manual Interim Report Tasks 2 to 6	February 15, 2007 May 31, 2007 August 15, 2007 July 15, 2007	May 15, 2007 * July 31, 2007 October 15, 2007 September 15, 2007
Task 7 Draft Final	June 30, 2007 September 15, 2007	August 31, 2007 November 15, 2007
Task 8 – Executive Summary Draft 4 to 6 page summary Final 4 to 6 page summary	June 30, 2007 August 31, 2007	August 31, 2007 October 31, 2007
Task 8 – 13 hour slides Draft Final	November 30, 2007 January 31, 2008	January 31, 2008 March 31, 2008