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Strategies for Managing Unknown Bridge Foundations

U.S. Department of Transportation
Federal Highway Administration

Research and Development
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FOREWORD

This report summarizes the problem of unknown bridge foundations and provides a strategy for managing this problem by characterizing the Nation's bridges over water with respect to key parameters such as average daily traffic, length, functional classification data, scour-related parameters, and other data contained in the National Bridge Inventory. The subset of bridges with unknown foundation conditions is characterized and quantified to determine their impact on scour evaluations presently in progress. A risk-based procedure has been developed to allow the bridge owner to manage bridges with unknown foundations. A model is provided to assess the need to determine the type and depth of foundation. The model allows the owner to prioritize bridges having the greatest urgency or economic benefit for determining attributes about the foundation and depth. Finally, a methods guide outlining measures that might be taken to determine foundation type and depth is presented.

Copies of this report are being distributed to each FHWA Region and each FHWA Division and State highway agency. Direct distribution is being made to the Division offices. This report will be of interest to highway engineers involved in bridge and scour studies. Additional copies may be purchased from the National Technical Information Service (NTIS), 5285 Port Royal Road, Springfield, VA 22161.

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Office of Engineering and Highway
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NOTICE

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16. Abstract
State highway agencies have plans and records on the type and depth of foundations for most Federal-aid highway system bridges, though, not nearly as much data exists for off-system bridges. Information on type and depth of the foundations, however, is essential for conducting scour evaluations in order to respond to criteria for the National Bridge Inspection Standards (NBIS). This report summarizes the problem of unknown foundations and provides a strategy for managing the problem of unknown foundations.

This report characterizes the nation's bridges over water with respect to key parameters such as average daily traffic (ADT), length, functional classification data, scour related parameters, and other data contained in the National Bridge Inventory (NBI). The subset of bridges with unknown foundation conditions is characterized and quantified in order to determine their impact on scour evaluations presently in progress. A risk based procedure has been developed to allow the bridge owner to manage bridges with unknown foundations. A model is provided to assess the need to determine the type and depth of foundation. The model allows the owner to prioritize bridges having the greatest urgency or economic benefit for determining attributes about the foundation type and depth. Finally a methods guide outlining measures that might be taken to determine foundation type and depth is presented.

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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yards	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
NOTE: Volumes greater than 1000 l shall be shown in m^3.				
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5(F-32)/9 or (F-32)/1.8	Celcius temperature	°C
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.71	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact)		
°C	Celcius temperature	1.8C + 32	Fahrenheit temperature	°F
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

* SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380.

(Revised September 1993)

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CHAPTER 1. INTRODUCTION

BACKGROUND

The renewal of our deteriorating highway infrastructure has become the focus of highway agencies in the 1990's. In particular scour related damage leading to bridge failure has captured recent attention.

Attention has been focused on identifying structures subject to scour damage in order to determine scour critical bridges. The August 26, 1988, Federal Register contains the final rule for revising 23 CFR 650, Subpart C of the National Bridge Inspection Standards (NBIS). This revision includes Item 113, Scour Critical Bridges, which identifies the current status of the bridge regarding its vulnerability to scour. The Technical Advisory on "Evaluating Scour at Bridges" October 21, 1991, is the implementing document. Prior to this revision, scour critical bridges were not identified and therefore would not receive the appropriate treatment.

Hydraulic Engineering Circular 18 recommends, in outline, a process to evaluate the vulnerability of existing bridges to scour by identifying in order the following[1]:

1. Bridges currently experiencing scour or that have a history of scour problems during the past floods as identified from owners records. This first-cut analysis does not require specific knowledge of bridge type, size, consequence of failure, etc. or availability of construction plans.

2. Bridges over streams with erodible beds with design features that make them vulnerable to scour, such as spread footings or short pile foundations, simple spans or non-redundant support, or constructed with inadequate waterway openings. This process would require specific knowledge of stream bed characteristics, bridge type, size, length, and configuration; and foundation type, size, and depth.

3. Bridges located on stream reaches with adverse flow characteristics. This process carried to its logical conclusion would also require specific knowledge of topography, stream bed characteristics, and bridge and foundation characteristics.

From the above, it is clear that items 2 or 3 cannot be evaluated without knowledge of the physical characteristics of the stream, bridge, and its substructure units. Bridges identified under item 1 as being critical cannot be subjected to the field and office evaluation required under the FHWA document without a knowledge of foundation type and depth.

In implementing these requirements, it has become apparent that plans, specifications, and construction records for a significant portion of the system are not available and therefore scour evaluations required to respond may be delayed or require the expenditure of substantial funds. The actual total magnitude of the problem associated with unknown bridge foundations has been preliminarily determined (1991) from raw data obtained in connection with reporting procedures developed in compliance with the National Bridge Inspection Program pursuant to an FHWA memo dated February 5, 1990, where bridges are being preliminarily assessed as low scour risk, scour susceptible, or with unknown foundations.

It therefore appears that the development of a strategy and procedure for bridge owners to manage the risk of not knowing the type, size, depth, configuration, or condition of a bridge foundation, is essential.

The determination of unknown foundations types in a practical, cost efficient manner is an important adjunct to the implementation of required scour evaluations. Available and emerging methods have been catalogued in this report to provide a data base and selection matrix enabling the responsible agency to implement site specific programs within the scope of the overall strategy developed.

PURPOSE AND SCOPE

The objective of this study is to develop a rational strategy and procedures for agencies to use in managing the risks associated with not knowing the type, depth, configuration, or condition of a bridge foundation. The results of these studies may be used by bridge owners to set scour evaluation priorities to insure that bridges most urgently in need of data on foundations are evaluated first.

Specifically, in order to attain this objective, this study included:

- The characterization of the problem nationwide using the National Bridge Inventory (NBI) database, and/or State databases to develop a statistical profile and define the extent and severity of the problem.

- Development of a practical guide outlining both conventional and new technological methods of determining the type and depth of unknown foundations. The guide provides criteria for evaluating costs, benefits, and effectiveness of each. This guide may further establish the need, potential, and direction for future research on instrumentation and detection methods.

- Development of a rational and practical strategy for assessing and managing the risk of not knowing the type and depth of foundations of a bridge for which a scour evaluation is required. This procedure is intended for use by agencies for determining which bridges most urgently need foundations data for scour evaluations.

CHAPTER 2. PROBLEM DEFINITION

BRIDGES OVER WATER

Public bridges in the Nation's Highway System are inventoried in the National Bridge Inventory (NBI). A bridge as defined in the Code of Federal Regulations is: "A structure including supports erected over a depression or an obstruction, such as water, highway or railroad and having a track or passageway for carrying traffic or other moving loads, and having an openway measured along the center of the roadway of more than 20 ft between undercopings of abutments or spring lines of arches, or extreme ends of openings for multiple boxes; it may also include multiple pipes, where the clear distance between openings is less than half of the smaller continuous opening."

The NBI record for each bridge is a significant collection of key features used to identify and characterize the type, usage, size, location, and condition of each structure. At present the total structure count in the NBI is roughly 577,000 structures.

The universe of bridges which are of interest for this study consists of structures over water, excluding culverts since they have much lower risk of severe scour leading to potential collapse or need for extensive rehabilitation. Structures over water excluding culverts represent roughly 391,000 structures or 67.5 percent of the total inventory. The balance represents structures over roadways, railroads, culverts, other. It should be noted that this national average may be misleading as significant variations exist from State to State as shown on figure 1, where only 16 percent of the States mirror this national average. Therefore significant variation must be anticipated nationwide.

The universe of bridges over water (391,000) is primarily located in rural (87 percent) areas and can also be broken down based on functional classifications as shown on table 1 below.

Table 1. Distribution of bridges over water based on functional classification.

Classification	% of Total over Water
Interstate, Freeways and Principal Arterials	4.4
Other Principals Minor Arterials, Major Collectors	34.9
Minor Collectors, Local	60.7

Table 5. Bridges over water, structure length in feet.

Classification	% of Total	% 20-50	% 51-125	% 126-250	% > 250
Interstate, Freeways Principal Arterials	4.4	5.0	22.8	32.5	39.7
Other Principals, Minor Arterials, Major Collectors	34.9	28.2	35.5	21.4	14.9
Minor Collectors Locals	60.7	57.7	32.6	7.6	2.1

The data from tables 1 through 5 suggests the following predominant characteristics for bridges over water:

- Predominantly rural: 87 percent
- Predominantly minor collectors, local: 61 percent
- Less than 50 yrs in service: 65 percent
- ADT less than 100: 40 percent
- ADT less than 1000: 72 percent
- Simple span construction: 74 percent
- Span length less than 50 ft: 45 percent
- Span length less than 125 ft: 78 percent

The potential vulnerability of a structure to scour events may be roughly indicated by examining certain ratings as contained in the NBI such as substructure condition (NBI Code 60), condition of channel and channel protection (NBI Code 61), and condition of waterway adequacy (NBI Code 71). For an overview, tables 6 through 10 have been prepared to provide a national statistical picture.

Table 6. Substructure condition rating, bridges over water (Item 60).

Condition	% Affected
Failed to poor condition (0-4)	14.8
Fair to satisfactory condition (5,6)	32.7
Good to excellent condition (7,8,9)	52.4

Within each Substructure rating category, bridges over water may be further subdivided as a function of ADT as shown on table 7. The distribution suggests that bridges carrying high ADT's have the lowest percentage of poor substructure condition ratings, reflecting higher construction or maintenance practices. It should be noted that over 50 percent of structures with ADT's of less than 100 are characterized by substructure condition ratings of poor to failed.

Table 7. Substructure condition rating vs. ADT.

		% within each Rating Category			
		ADT			
Condition	% of Total	0-100	101-1000	1001-5000	>5000
Failed to poor condition (0-4)	14.8	56	27	11	6
Fair to satisfactory condition (5,6)	32.7	42	30	17	11
Good to excellent condition (7,8,9)	52.4	35	33	19	13

Examination of the Waterway Adequacy Rating (Item 71) suggests that overtopping frequency is slightly greater for those structures functionally classified as Minor Collectors, Local, where ADT's are low as shown on table 8.

When each Waterway Adequacy category is further examined as a function of ADT as shown on table 9, a pattern emerges suggesting that the greatest percentage of vulnerable structures are associated with Minor Collectors, Locals with low ADT's.

If age of structure is examined as a function of waterway adequacy, a general pattern suggesting older structure on Minor Collectors, Locals as being vulnerable emerges as shown on table 10, where the at risk percentage is considerably greater than their percentage on the whole system as shown on table 4.

The last indicator, channel and channel protection rating shown on table 11, suggests again that bridges on Minor Collectors, Locals have a slightly higher incidence of failed or eroded bank conditions.

Although the patterns suggested by tables 6 through 11 may be intuitively obvious, the actual quantification as a national average may not be. It should be further noted that significant deviations may exist from State to State although the general pattern should remain constant. Further, the merit of the raw data in the NBI varies considerably from agency to agency.

The economic loss associated with a structure being out of service whether by scour or any other event is a function of ADT and detour length. Table 12 indicates the detour length associated on a nationwide basis with functional classifications.

Table 8. Bridges over water, condition of waterway adequacy (Item 71).

Classification	% of Total	% Frequent overtopping of bridge deck	% Occasional overtopping of roadway	% Slight Chance of overtopping	% Slight to remote chance of overtopping	Remarks
Interstate, Freeways Principal Arterials	4.4	0.06	0.2	19.8	79.4	---------
Other Principals Minor Arterials Major Collectors	34.9	0.71	6.8	30.1	62.4	---------
Minor Collectors Local	60.7	5.4	26.1	22.1*	46.4	*Occasional overtopping with insignificant traffic delays

Table 9. Waterway adequacy vs. ADT as a percentage in each category.

Waterway Adequacy & ADT	Functional Classification		
	Interstate Freeways Principal Arterials	Other Principals, Arterials, Major Collectors	Minor Collectors, Local
1. Bridges subject to frequent overtopping of bridge deck	(0.06% of total)	(0.7% of total)	(5.4% of total)
ADT <100	1	54	80
101-1000	1	32	17
1001-5000	17	13	2.5
>5000	80	1	0.5
2. Bridges subject to occasional overtopping of roadway approaches	(0.2% of total)	(6.8% of total)	(26.1% of total)
ADT <100	1	22	71
101-1000	25	40	24
1001-5000	10	25	3
>5000	64	13	1
3. Bridges subject to slight chance of overtopping of roadways	(19.8% of total)	(30.1% of total)	(22.1% of total)
ADT <100	1	10	60
1001-1000	6	36	32
1001-5000	15	36	6
>5000	78	18	2
4. Bridges subject to slight to remote chance of overtopping	(79.4% of total)	(62.4% of total)	(46.4% of total)
ADT <100	1	6	55
101-1000	3	34	35
1001-5000	23	39	7
>5000	73	21	2

Table 10. Waterway adequacy vs. age.

Water Adequacy & Age	Functional Classification: Interstates, Freeways Principal Arterials	Other Principals Arterials Collectors	Minor Collectors Local
1. Bridges subject to frequent overtopping of bridge deck	(0.06% of total)	(0.7% of total)	(5.4% of total)
Age over 75 years	-	5	17
35 - 75 years	76	51	56
less than 35 years	24	44	27
2. Bridges subject to occasional overtopping of roadway approaches	(0.2% of total)	(6.8% of total)	(26.1% of total)
Age over 75 years	-	6	12
35 - 75 years	47	66	56
less than 35 years	53	28	32
3. Bridges subject to slight chance of overtopping roadways	(19.8% of total)	(30.1% of total)	(22.1% of total)
Age over 75 years	-	3	10
35 - 75 years	21	61	43
less than 35 years	79	36	47
4. Bridges subject to slight to remote chance of overtopping roadways	(79.4% of total)	(62.4% of total)	(46.4% of total)
Age over 75 years	3	2	8
35 - 75 years	11	45	35
less than 35 years	86	53	57

Table 11. Bridges over water, condition of channel and channel protection (Item 61).

Classification	% of Total	% Closed	% Near Collapse	% Failed Bank Protec.	% Bank Protec. Severely Undermined	% Bank Protec. Severely Eroded	% Widespread Minor Damage	% Minor Damage	% No Deficiencies
Interstate, Freeways Principal Arterials	4.4	-	-	0.4	1.0	4.0	15.0	29.8	49.4
Other Principals Minor Arterials, Major Collectors	34.9	0.04	0.2	1.1	4.1	8.3	20.6	29.4	36.1
Minor Collectors Local	60.7	0.3	0.2	1.5	6.5	12.0	23.9	29.1	26.2

Table 12. Detour length vs. functional classification.

Classification	% of Total over Water	Detour length in miles % within each category 0-4	5-10	11-99	>100
Interstate, Freeways Principal Arterials	4.4	81	9	7	3
Arterials, Major Collectors	34.9	42	24	30	4
Minor Collectors, Locals	60.7	60	26	8	6

The data presented defines a statistical picture of the systems bridges excluding culverts, over water. Because of the large size of the database, any subset should mirror the national average although not necessarily on a State by State basis.

BRIDGES WITH UNKNOWN FOUNDATIONS

At the inception of this study (summer of 1990) no hard data was available nationwide on the magnitude of this problem, although it was understood that a significant percentage of structures over water lacked foundation information to complete scour evaluations.

The February 5, 1990, memo from the FHWA Office of Engineering to Regional Federal Highway Administrators initiated a screening process to determine the magnitude of the scour and unknown foundations problem on a State by State basis and required completion by March 31, 1991, of this initial screening.

The data developed by March 31, 1991, represented an 80 percent screening effort for the on-system and a 51 percent overall screening effort. The raw figures developed from existing State databases suggested that approximately 10 percent of the on-system bridges, including culverts, lacked foundation information and 35 percent of the off-system bridges.

Examination of the raw data and discussions with various States strongly suggested a widespread lack of uniformity in determining what constitutes an unknown foundation.

In some States, bridges not categorized as scour susceptible or low risk or when foundation type was known but not its elevation were coded in the unknown foundation category. In other States, if foundation type was known (piles, spread footings), but the elevation not known, the bridges were deemed to be with known foundation, especially if pile founded. It should be noted that for structures with pile supported foundation units (piers, abutments), pile tip or driving record information is seldom recorded in the database, although it might exist in agency records somewhere.

Table 13. Summary of scour evaluations projections based on March 31, 1991 data.

FHWA Region	Total Bridges over Water	% of National Total	Total Screened	% Screened	Total Coded Unknown Fnd. to Date	% Unknown of Screened Total	Remarks
Region 1	28,533	5.9	18,393	64.4	4,737	25.7	
Region 3	36,472	7.6	33,503	91.9	3,916	11.7	
Region 4	95,953	19.9	89,964	93.8	22,293	24.8	Projection high. NC codes as unknown foundation all structures not scour susceptible or low risk
Region 5	86,915	18.0	62,439	71.8	13,821	22.1	Projection high. Indiana codes as unknown if not low risk or scour susceptible
Region 6	93,148	19.3	53,806	57.8	13,555	25.2	Projection suspect. Low level of data
Region 7	81,683	16.9	62,549	76.6	21,775	34.8	Projections suspect. Nebraska and Missouri use this category for susceptible structures not low risk or scour
Region 8	23,785	4.9	15,978	67.2	7,137	44.7	
Region 9	21,422	4.4	21,422	100	1,889	8.8	
Region 10	15,102	3.1	14,125	93.5	1,367	9.7	
Totals	483,013	100	372,179	77	90,490	24.3	

Table 14. Screened projected unknown foundations.

FHWA Region	Total Bridges Water	Culverts over 20 feet	Bridges Only	% Screened	% Reported Unknown of Screened Total	Projected % Bridges Only	Projected Unknown
Region 1	28,533	3,083	25,450	64.4	25.7	25	6,263
Region 3	36,472	4,952	31,520	91.9	11.7	12	3,782
Region 4	95,953	24,292	71,661	93.8	24.8	25	17,915
Region 5	86,915	14,582	72,333	71.8	22.1	25	18,083
Region 6	93,148	26,578	66,570	57.8	25.2	17	11,317
Region 7	81,683	14,594	69,089	76.6	34.8	17	11,405
Region 8	23,785	3,449	20,336	67.2	44.7	30	6,101
Region 9	21,422	6,499	14,923	100	8.8	9	1,343
Region 10	15,102	327	14,775	93.5	9.7	10	1,478
Totals	483,013	97,073	385,940	77	24.3	-	77,687

The raw data therefore requires significant additional screening and projections on a uniform basis to be useful. If the data is simply projected on the basis of the March, 1991, responses, it would appear that 24 percent of the overall system would be categorized as unknown foundations, or nearly 116,000 structures as shown on table 13.

If a uniform definition of unknown foundation is adopted to include only those structures excluding culverts, whose type and depth of foundation is unknown, the percentage would be significantly reduced to possibly as low as 20 percent of the total system. The on-system bridges with unknown foundations under this definition are not likely to exceed 15 percent or 21,500 structures, mostly in States with older urban infrastructures or where cities, counties, or other Agencies originally planned or constructed these facilities.

The off-system bridges with unknown foundations under this definition are difficult to project, but are estimated at no more than 21 percent of the total or roughly 56,000 structures. A significant percentage is likely to be low risk with respect to scour, after a field review, even without knowledge of the foundation. Therefore, it is anticipated that the requirement of determining the actual foundation type or elevation will be significantly reduced. Table 14 represents a screened projection under this revised definition based on limited qualitative judgments made based on the reported figures.

CHARACTERISTICS OF BRIDGES AT GREATEST RISK

A statistical picture nationwide can be developed by examination of the characteristics of structures as a function of waterway adequacy rating (Item 71, NBI). Bridges rated as closed or with waterway adequacy ratings of two, indicating frequent to occasional overtopping of the deck and roadway approaches, can be expected to have (or had) the greatest incidence of failure due to scour events. They can be divided based on functional classification as shown on table 15.

Table 15. Bridges with inadequate waterway ratings or failed.

Classification	% Failed or Inadequate
Principal Arterials	0.02
Other Principal, Minor Arterials	7.3
Minor Collectors, Locals	92.5

From the above, it is clear that Minor Collectors and Locals constitute the overwhelming majority of failed or at great risk structures, to a percentage far greater than their total over water, of 60.7 percent, as shown on table 1.

Further subdivision of this at risk group as a function of ADT, length of structure, age and a detour length is presented on table 16.

Table 16. Minor collectors, locals with inadequate waterway ratings or failed.

Category	% Failed or Inadequate	% Over Water
ADT		
0-100	80	62
101-100	17	31
1001-5000	2.5	6
>5000	0.5	1
Length		
20-50	68	58
50-125	27	33
125-250	4	8
>250	1	1
Age		
More than 75 yrs	17	10
35 to 75 yrs	56	43
Less than 35 yrs	27	47
Detour Length		
0-4 miles	58	60
5-10 miles	31	26
11-99 miles	6	8
>99 miles	5	6

Examination of the data from table 16 suggests that the at-risk group can be generally characterized as older, simple span structures less than 50 ft in length with ADT's of less than 100. This would strongly suggest that the economic impact of failure is comparatively low in this higher risk group.

FHWA Report RD-75-87 studied the mode of failure or damage associated with certain specific flood events and presented costs associated with repair and reconstruction.[2] The data is based on Emergency Relief files for the 1969, 1970, and 1972 floods. The cause of damage sustained for values in excess of $1,000 per structure are statistically summarized on table 17.

Table 17. Cause of damage.

Cause	% of Cases
Riverbed Change	6.9
Flow Change	29.6
Flowpath Deficiency	38.8
Floating debris	20.0
Structural deficiency	4.7

The major causes of collapse or damage attributable to scour events are stream or flow associated. The location of damage is statistically summarized in table 18.

Table 18. Location of damage.

Location	% of Cases
Superstructure	14.9
Pier	24.5
Abutment	71.8
Approach Road	43.2

Further, the data indicates that 25.6 percent of the studied bridges were classified as "collapsed", and judging from the reconstruction costs tabulated, approximately 33 percent required complete reconstruction. The cost data further suggests that the reconstructed bridges were longer and wider than the original structure, on average having a surface area (length times width) 2.5 to 3 times greater. Of the roughly 67 percent of structures sustaining repairable damage, the average cost of repair was equivalent to 15 to 20 percent of the then current average cost of replacement with a bridge of equivalent length and width.

The data further suggests that short span bridges (less than 50 ft) are somewhat less susceptible to collapse than would be indicated by their numbers.

A more recent study of bridge failures nationally since 1950 indicates that 60 percent of all failures are attributable to "hydraulics" although the specific hydraulic deficiency is not detailed.[3] In the same time period the percentage in New York State was 40.[2]

In both cases the majority occurred on rural off-system roads generally mirroring the distribution on the total system as a function of functional classification, length, etc.

CHARACTERISTICS OF BRIDGES WITH UNKNOWN FOUNDATIONS

The significant size of this universe when compared to the universe of all bridges over water would strongly suggest that they would have comparable characteristics with respect to ADT, span length, substructure condition, waterway adequacy, and channel protection ratings, where this category is a significant percentage of bridges over water within each State. Where percentages are small, their characteristics will be random.

The above hypothesis was checked by examining in detail bridges classified as having unknown foundations in the States of Maryland and Pennsylvania which represent a fair range in the universe size in States with older infrastructure. Maryland has 4 percent unknown foundations and Pennsylvania has 12 percent unknown foundations.

The Pennsylvania unknown data has been analyzed and compared to the total statewide data. This serves as a reality check to the hypothesis that where significant percentages of unknown foundations for bridges exist, their distribution with respect to key features should mirror the statewide distribution for all bridges over water. The comparative data is presented in tables 19 through 22.

Table 19. Unknown foundations vs. functional classification in Pa.

Classification	% Unknown	% Over water
Interstate, Freeways, Principal Arterials	1.5	3.2
Arterials, Major Collectors	32.4	34.6
Minor Collectors, Locals	66.1	62.2

Table 20. Unknown foundations vs. ADT in Pa.

ADT	% Unknown	% Over water
0-100	26.5	19.2
101-1000	33.2	38.8
1001-5000	20.6	25.0
>5000	19.7	17.0

Table 21. Unknown foundations vs. length in Pa.

Length	% Unknown	% Statewide over water
20-50	65.4	58.5
51-125	26.3	26.5
126-250	6.0	8.1
>250	2.3	6.9

Table 22. Unknown foundations vs. age in Pa.

Age	% Unknown	% Statewide over water
Over 75 yr old	19.7	10.5
35 - 75 yr old	65.3	57.1
Less than 35 yr old	15.0	32.3

Although the hypothesis is reasonably validated, there appears a trend to indicate that the unknown category comprises a greater degree of older (more than 75 yr old) and shorter (20 to 50 in length) structures than the total statewide or total nationwide averages. As the percentage of unknowns in each state increases they should more closely mirror statewide averages.

CHAPTER 3. DEVELOPMENT OF A RATIONAL STRATEGY

GENERAL

The objective of this study is the development of a rational and practical strategy for assessing and managing the risk of not knowing the type and depth of bridge foundations potentially subject to scour. The methodology developed in this study sets priorities based on factors developed from the NBI or State Bridge Inventory databases. Further refinement can be obtained from local historical knowledge and features that can be observed through site visits.

The relative risk of scour failure or heavy damage for bridges with unknown and presumed inadequate foundations is calculated as the product of the cost associated with failure and or heavy damage and the relative probability of the occurrence. The method is based on data (much of which is subjective) contained in the National Bridge Inventory (NBI). The relative risk determines the ranking of bridges for foundation data gathering in support of scour evaluation; high calculated risks could vanish if substantial foundation information is developed or inferred.

The key is the ranking. High risk situations are addressed and resolved first. Information about these situations is more valuable and necessary to minimize potential losses.

Existing formal or informal state ranking methodologies developed by Maryland, North Carolina, Colorado, and New York were reviewed to determine ranking factors considered and relative importance factor weights. The following key parameters have been identified from this review:[9,10,11]

- ADT.
- Functional classification.
- Type of span (simple, continuous).
- Type of foundation (pile, spread, unknown).
- Condition ratings (waterway adequacy, channel protection, substructure rating).
- Field scour evaluations.

RISK BASED MODEL

The risk based model developed in this study considers many of the items identified in the described plans as well as other applicable data found in NBI to prioritize structures according to relative risk. It is equally appropriate, conceptually, where foundations are known. The risk is calculated as the product of the probability of scour failure or heavy damage and the losses associated with such an event. Risk is the expected value of the loss. The three categories of costs used in the model include:

- Rebuilding cost.
- Additional running cost.
- Additional time cost.

Property damage, injury, and death costs can be high but when weighted by their probability, their risks were determined to be negligible compared with the other risks. The model is shown below, with corresponding NBI item numbers for applicable parameters. A detailed discussion follows.

$$\text{Risk} = KP\ [\underbrace{C_1\ WL}_{\text{Rebuilding Cost}} + \underbrace{C_2\ DAd}_{\text{Running Cost}} + \underbrace{(C_3\ O(1 - T/100) + C_4\ T/100)\ DAd/S}_{\text{Time Cost}}] \qquad (1)$$

Where:

Risk	=	risk of scour failure, \$/year
K	=	Risk adjustment factor based on foundation type and type of span (based on NBI items and where available from more developed databases, foundation information)
P	=	probability of failure (based on NBI items 26, 60, 61, 71), year^{-1};
C_1	=	unit rebuilding cost (\$/sf.);
W	=	bridge width, ft (NBI item 52);
L	=	bridge length, (NBI item 49);
C_2	=	cost of running vehicle (\$0.25/mi);
D	=	detour length, mi (NBI item 19);
A	=	ADT (NBI item 29);
d	=	duration of detour, days (based on ADT-NBI item 29);
C_3	=	value of time per adult in passenger car, \$7.05/h (1991);
O	=	average occupancy rate, 1.56 adults;
T	=	average daily truck traffic, percent of ADT (NBI item 109);
C_4	=	value of time for truck, \$20.56/h (1991); and
S	=	average detour speed, 40 mi/h.

Risk Adjustment Factor

This factor permits downward risk adjustments based upon knowledge of the structural and/or foundation design. The equation is:

$$K = K_1 K_2,$$

Where:

K_1 = bridge type factor obtained from NBI

and

K_2 = foundation type factor, information about which may be contained in State inventories but not in the NBI.

The values presently recommended for K_1 are:

1.0 = Simple spans

0.8 = Continuous spans with lengths of less than 100 feet

0.67 = Rigid continuous spans with lengths on excess of 100 ft

This factor adjusts to reflect the benefit of structural continuity which can compensate for loss of intermediate supports. The factors are subjective, based on a limited delphic survey and data developed in FHWA RD-85-107, Tolerable Movement Criteria for Highway Bridges.[13] The influence of actual rigidity, type of structure, etc. has significant effects on the tolerable movement criteria, which may be defined as an increase in maximum stress to a point below yield, therefore precluding the collapse case.

The values presently recommended for K_2 should be developed for both abutment and pier condition using the largest for analysis. They are:

1.0 = <u>Unknown</u> foundation or spread footing on erodible soil above scour depth. Pier footing top visible or 1 to 2 ft below stream bed.

0.8 = Pile foundation of <u>unknown</u> length or when length is known and less than 19 ft or all wood pile foundations.

0.5 = Pile foundations with lengths in excess of 20 ft below present stream bottom.

0.2 = Foundations on massive rock.

These factors are again subjective, reflecting the relative values assigned to this item by the various existing ranking criteria previously described and should be revised or adjusted using local experience or further forensic studies. It should be noted that even structures supported on massive rock foundations may still suffer damage due to inadequate waterway openings or other causes. Therefore the risk adjustment factor cannot by definition be zero in a dollar based risk analysis.

Probability of Failure

The probability of scour failure is estimated based on waterway adequacy (NBI item 71), functional classification (NBI item 26), substructure condition (NBI item 60), and channel protection (NBI item 61). The waterway adequacy and functional classification are used to determine the overtopping frequency, as described in the NBI recording guide and shown in figure 2. For example, a bridge with a functional classification code of 17 (Urban Collector) and a waterway adequacy code of 4 has an overtopping frequency of 3 to 10 yr (or an annual overtopping probability of approximately 0.2). The overtopping frequency is important since bridges can be more susceptible to scour failure during flood events.

If the overtopping frequency is known or can be estimated, say 0.01, one also can estimate the frequency that the bridge opening is full of water. This full condition also represents a good estimate of maximum depth since higher flow will be accommodated by embankment overtopping without large depth increases. The logic derives the frequencies of less than full flow depths using USGS regression equations and proportionalities implied by the Manning's normal flow equation.

The logic is followed because shear stress is proportional to depth, and scour is proportional to shear stress. Thus, one has maximum scour potential at full flow depth and less potential at lesser depths.

Functional Class: Item 26	CODES	Waterway Adequacy: Item 71 CODES 0	1	2	3	4	5	6	7	8	9	N
Principal Arterials-Interstates	01,11			Ø F	Ø	Ø	Ø	S	S	S	R	N
Freeways or Expressways	12											
Other Principal Arterials	02,14	Bridge Closed	Unused	F	Ø	Ø	Ø	S	S	S	R	N
Minor Arterials	06,16											
Major Collectors	07,17											
Minor Collectors	08			F	F	Ø	Ø	Ø	S	S	R	N
Locals	09,19											

Overtopping Frequency	Annual Probability	Return Period (yrs)
N None	0	Never
R Remote	.01	> 100
S Slight	.02	11 to 100
Ø Occasional	.2	3 to 10
F Frequent	.5	< 3

Figure 2. Annual probability of flooding bridge opening.

The USGS regression equations for Virginia are:[13]

$$Q_2 = 25.2\ A^{0.83}\ S^{0.26}\ RF \tag{2}$$

$$Q_5 = 52.2\ A^{0.80}\ S^{0.25}\ RF \tag{3}$$

$$Q_{10} = 81.3\ A^{0.78}\ S^{0.24}\ RF \tag{4}$$

$$Q_{50} = 198\ A^{0.74}\ S^{0.24}\ RF \tag{5}$$

$$Q_{100} = 269\ A^{0.73}\ S^{0.21}\ RF \tag{6}$$

where:

A	=	drainage area
S	=	slope, feet per mile; and
RF	=	regional factor

Normal depth of channels using Manning's equation in both numerator and denominator and using the hydraulic radius as the depth gives:

$$\frac{Q}{Q_f} = \left[\frac{D}{D_f}\right]^{1.66} \quad \text{or} \tag{7}$$

$$\frac{D}{D_f} = \left[\frac{Q}{Q_f}\right]^{0.6} \tag{8}$$

where:

Q_f = full flow (for which we attribute a frequency associated with overtopping) and $Q < Q_f$ and D/D_f is the depth ratio.

Thus, it can be reasoned that if the overtopping frequency is associated with Q_{100} (1 percent) that the fraction full frequency for Q (2 percent) can be calculated as:

$$\frac{D}{D_f} = \left[\frac{Q_{50}}{Q_{100}}\right]^{0.6} \tag{9}$$

This logic was applied to (Q/Q_f) ratios computed using the USGS Virginia equations for typical slope ranges of 0.2 percent to 1.5 percent and area ranges of 1 to 500. They will be equally valid for other states as the ratios would remain essentially constant.

For example, using a Q_2/Q_{100} flow ratio of 0.14 implies a depth ratio $D_2/D_{100} = 0.14^{0.6} = 0.31$. If the bridge waterway adequacy and functional classification indicate remote overtopping frequency (once every 100 yr), the annual probability of the waterway opening being full (depth = full depth) is 0.01. The depth ratio indicates that in a given year there is a 50 percent chance (D_2) of the waterway being 31 percent full (D_2=$0.31D_{100}$=$0.31D_{full}$).

This methodology was applied to develop the depth distributions for remote (0.01), slight (0.02), occasional (0.2), and frequent (0.5) overtopping as shown in figures 3, 4, 5, and 6, respectively.

It should be noted that a number of State databases contain detailed information as to stream velocity and depth of flow for each structure. This detailed information can be used to enhance the inferred probabilities shown on figure 2.

For example, a flood flow of <u>1500 cfs</u> is known to have a depth of <u>12 ft</u> with a bridge opening having a clearance of <u>15 ft</u>, and furthermore, the 1500 cfs flood is the <u>5 yr storm</u>. Therefore, one knows that the 5 yr storm (annual probability of 0.2) does not cause overtopping. This knowledge can be used to adjust the overtopping frequency codes shown in figure 2: None, Remote, Slight, Occasional, or Frequent.

A scour vulnerability rating for bridge foundations was developed based on the substructure code (NBI item 60) and the channel protection code (NBI item 61). The ratings range from 0 to 9 with the lower numbers indicating higher vulnerability. Definitions for the scour vulnerability are shown in table 23 and the rating development is shown in figure 7. For example, if item 60 is rated 3 (serious scour) and item 61 is rated 4 (undermined bank), the scour vulnerable bridge rating is 3 (scour vulnerable--unstable foundations). This scour vulnerable bridge rating is used where foundations are unknown (item 113 is coded 6 or is not included) by inferring scour vulnerability based on substructure condition and channel condition.

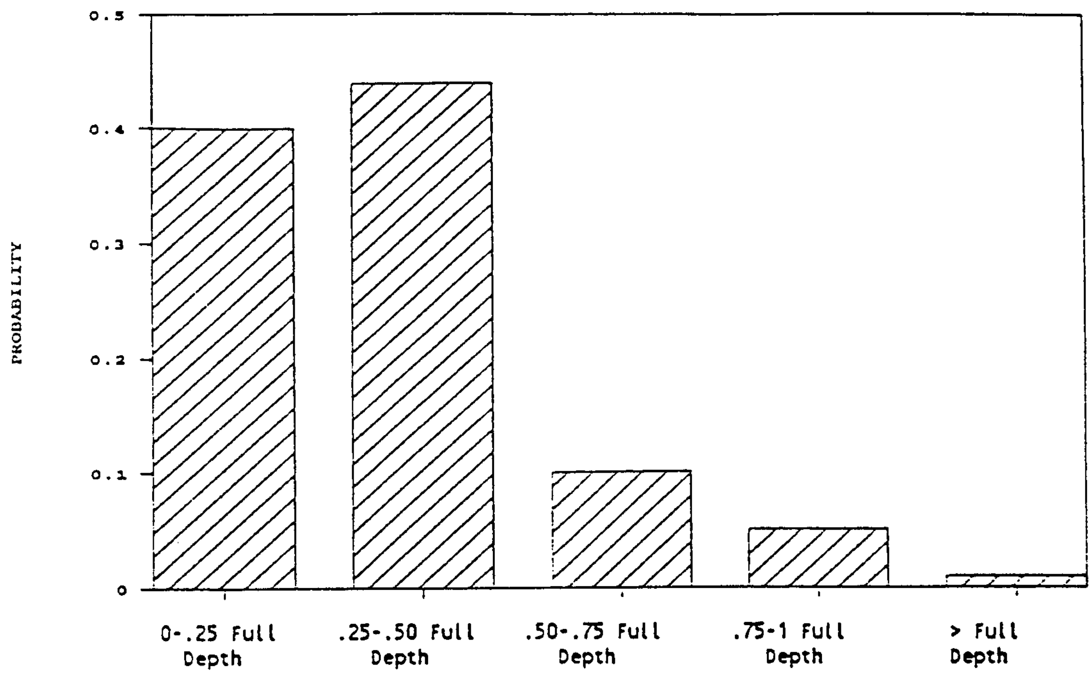

Figure 3. Remote overtopping depth distribution.

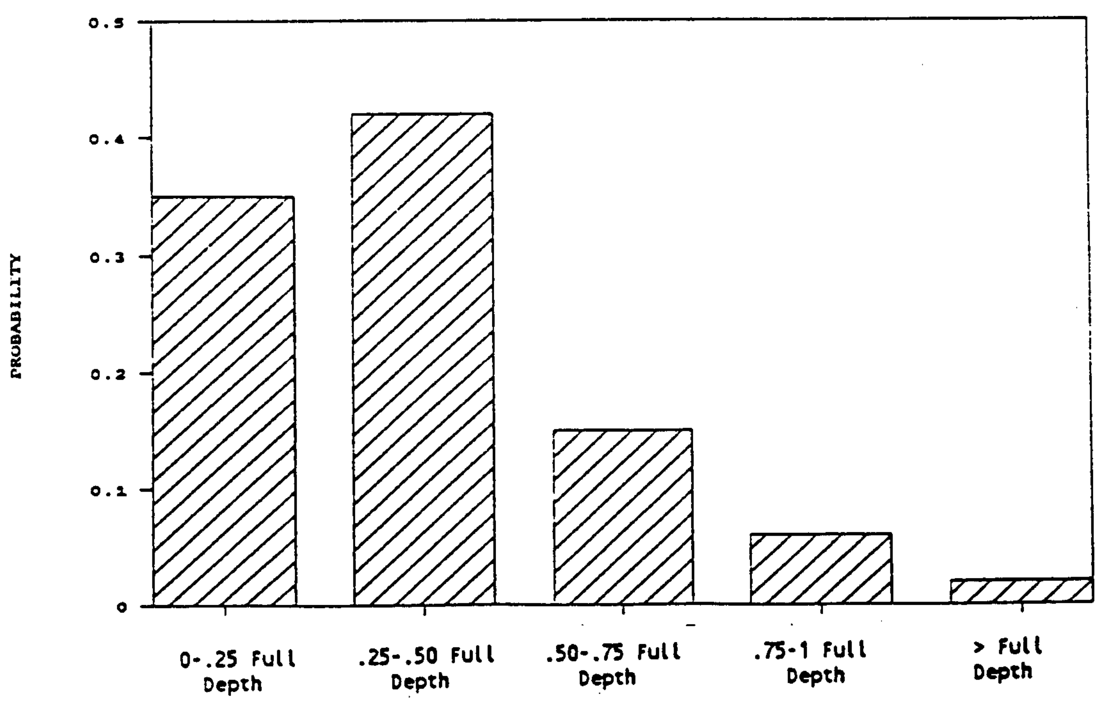

Figure 4. Slight overtopping depth distribution.

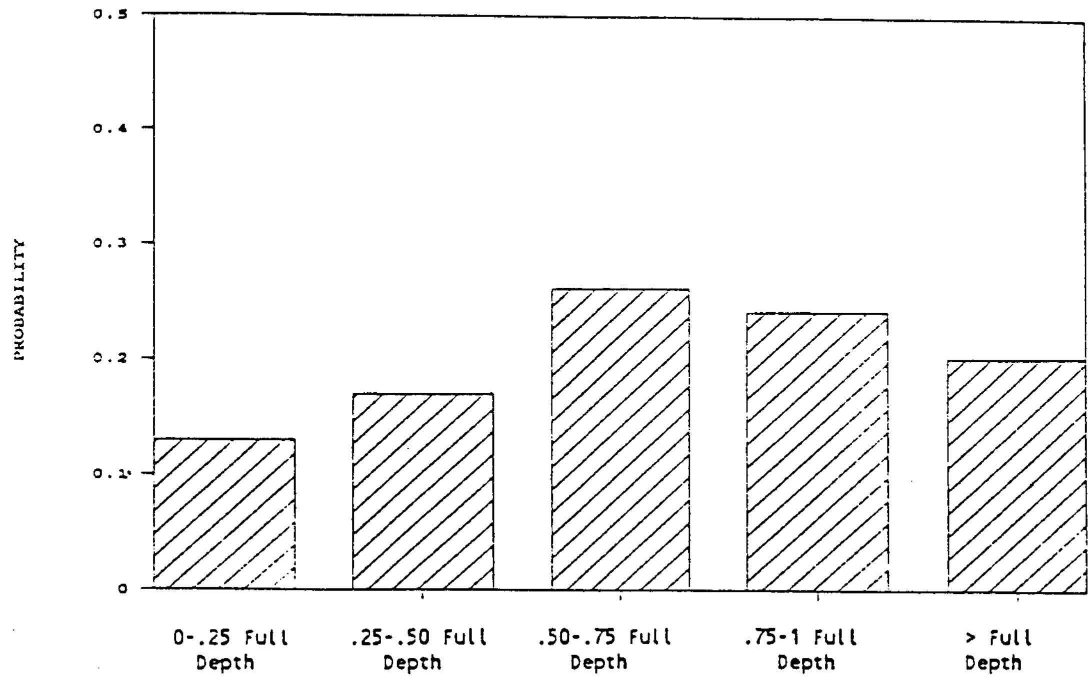

Figure 5. Occasional overtopping depth distribution.

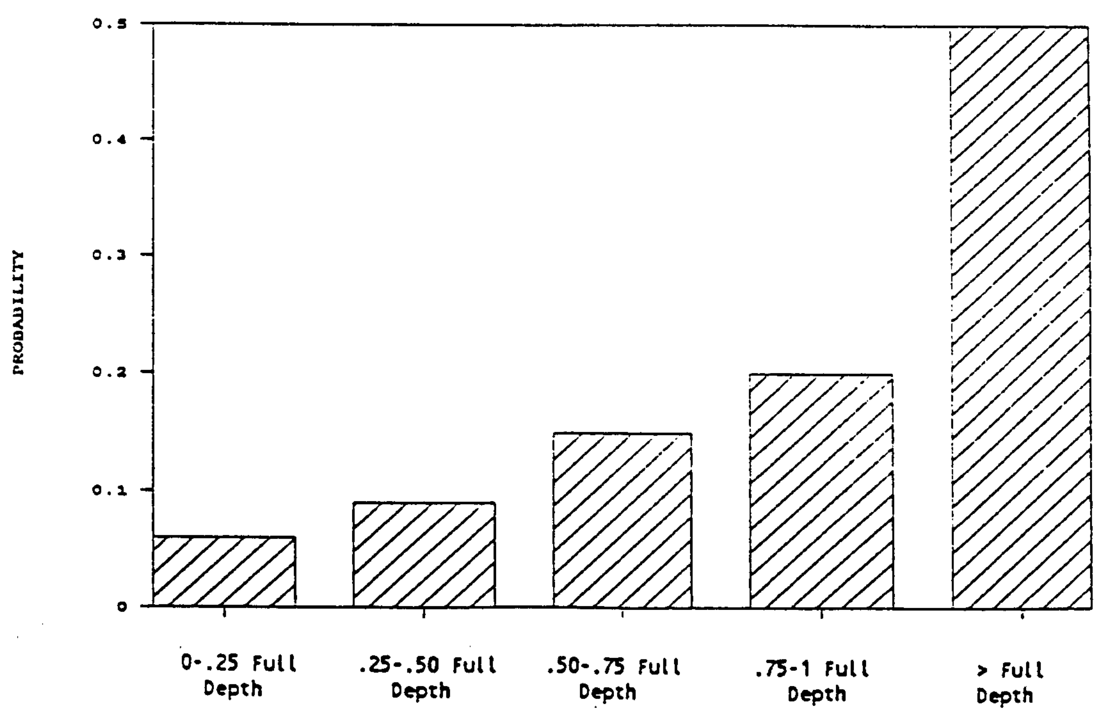

Figure 6. Frequent overtopping depth distribution.

Table 23. Scour vulnerable bridges determined by channel protection (Item 61) and substructure condition (item 60).

0	Scour vulnerable - bridge failed
1	Scour vulnerable - bridge closed
2	Scour vulnerable - immediate action required
3	Scour vulnerable - unstable foundations
4	Scour vulnerable - action required
5	Fair condition - minor damage to channel and/or substructure
6	Satisfactory condition - minor deterioration of channel and/or substructure
7	Good substructure condition, minor channel problems
8	Very good substructure condition, minor to no channel movement noted
9	Excellent condition - no deficiencies
N	Not applicable

A subjective scour conditional failure probability distribution conditioned to figure 7 was developed based on scour vulnerability and the depth distribution as shown in table 24. For example, if the scour vulnerability rating is 5 (fair condition) and the annual maximum depth is half to three-quarters full, the annual probability of failure is 0.08. These probabilities are based on conservative assumptions about the foundations, that is that the foundation is of the maximum risk type, a spread foundation on erodible soil potentially within the scour depth.

With the probability of failure given scour vulnerability and depth, and the probability of depth given overtopping frequency, the probability of failure given overtopping frequency and scour vulnerability was determined as follows:

$$P\ (F|(OT \text{ and } SV)) = \sum_{D} P\ (D|OT)\ P\ (F|(SV \text{ and } D) \qquad (10)$$

where:

F = failure;
OT = overtopping frequency;
SV = scour vulnerability; and
D = dimensionless depth.

Table 24. Probability of failure given depth and scour vulnerability.

Scour Vulnerability (items 60 and 61)	Depth Range* 0 - .25	.25 - .50	.50 - .75	.75 - 1.0	> 1.0
0 (Bridge failure)	1.00	1.00	1.00	1.00	1.00
1 (Bridge closed)	1.00	1.00	1.00	1.00	1.00
2 (Extremely vulnerable)	0.25	0.40	0.55	0.70	0.88
3 (Unstable foundations)	0.14	0.20	0.30	0.45	0.65
4 (Action required)	0.06	0.10	0.15	0.26	0.41
5 (Fair condition)	0.01	0.04	0.08	0.16	0.27
6 (Satisfactory condition)	0.002	0.01	0.03	0.08	0.18
7 (Good condition)	0.002	0.002	0.01	0.03	0.10
8 (Very good condition)	0.002	0.002	0.002	0.01	0.05
9 (Excellent condition)	0.002	0.002	0.002	0.002	0.01
N (Not over water)	0.002	0.002	0.002	0.002	0.002

*Dimensionless depth expressed as depth divided by full depth.

(Use if Item 113 is coded 6 or not included)

Channel Protection (Item 61)	Substructure Condition (Item 60): failed 0	bridge closed - imminent failure 1	critical scour 2	serious scour 3	advanced scour 4	minor scour 5	minor deterioration 6	good condition 7	very good condition 8	excellent condition 9	not applicable N
0 failure	0	0	0	0	0	0	0	0	0	0	0
1 failure	0	1	1	1	1	1	1	1	1	1	1
2 near collapse	0	1	2	2	2	2	2	2	2	2	N
3 channel migration	0	1	2	2	3	4	4	4	4	4	N
4 undermined bank	0	1	2	3	4	4	5	5	6	6	N
5 eroded bank	0	1	2	3	4	5	5	6	6	7	N
6 bed movement	0	1	2	3	4	5	6	6	7	7	N
7 minor drift	0	1	2	3	4	6	6	7	7	8	N
8 stable condition	0	1	2	3	4	6	7	7	8	8	N
9 no deficiencies	0	1	2	3	4	7	7	8	8	9	N
N not over water	0	1	N	N	N	N	N	N	N	N	N

Figure 7. Scour vulnerable bridges.

The above expression weights failure over the five depth ranges, eliminating depth as a variable. The result is an operational probability estimate. These operational failure probabilities are presented in table 24 and represent the probability used in the risk computation since they can be directly obtained from NBI data elements.

The procedure therefore consists of obtaining, first, substructure condition (item 60) and channel protection (item 61) and determining the scour vulnerability. Second, the using of overtopping frequency (items 26, 71 as related by figure 2) is obtained, and using table 25 the probability of annual failure is estimated given overtopping and vulnerability. The subjective elements in this process are in the definition of table 24 (failure given depth and vulnerability) and in the definition of vulnerability, figure 7.

Failure and/or Heavy Damage Costs

The rebuilding cost is estimated as a function of the bridge area which is calculated as the product of the bridge width (NBI item 52) and the bridge length (NBI item 49). A constant rebuilding cost of $60 per square ft can be assumed, or better State information substituted, yielding the following:

$$\text{Rebuilding Cost (\$)} = C_1 \, WL \tag{11}$$

where:

C_1 = rebuilding cost, ($60/ft^2, default cost)
W = bridge width, ft
L = bridge length, ft

A failure or heavy damage cost equal to the current rebuilding costs based on the original length and width can be justified on the basis of the data developed by Chang, namely:[4]

- Failure is estimated to occur approximately one third of the time at a rebuilding cost 2 to 2.5 times current costs based on the original length and width.
- Heavy damage occurs two thirds of the time at a repair cost 15 to 20 percent of current costs of rebuilding a structure of the same length and width.
- The average cost is therefore approximately equal to the current rebuilding cost of a structure of equal size for the purpose of this ranking analysis.

The additional running cost per vehicle is estimated currently as $0.25/mi to yield a total cost as follows:

$$\text{Running Cost (\$)} = C_2 \, DAd \tag{12}$$

Table 25. Probability of failure given overtopping frequency and scour vulnerability.

$$P(\text{Failure} \mid \text{Overtopping Frequency and Scour Vulnerability}) = \sum_{\text{Depth}} P(\text{Depth} \mid \text{Overtopping Frequency})\, P(\text{Failure} \mid \text{Scour Vulnerability and Depth})$$

Scour Vulnerability (items 60 and 61)	Overtopping Frequency (items 26, 71)			
	Remote (0.01)	Slight (0.02)	Occasional (0.20)	Frequent (0.50)
0 (Bridge failure)	1.00	1.00	1.00	1.00
1 (Bridge closed)	1.00	1.00	1.00	1.00
2 (Extremely vulnerable)	0.37	0.40	0.59	0.71
3 (Unstable foundations)	0.20	0.22	0.37	0.49
4 (Action required)	0.10	0.11	0.21	0.29
5 (Fair condition)	0.04	0.05	0.12	0.18
6 (Satisfactory condition)	0.01	0.02	0.06	0.11
7 (Good condition)	0.002	0.01	0.03	0.06
8 (Very good condition)	0.002	0.002	0.01	0.03
9 (Excellent condition)	0.002	0.002	0.002	0.01
N (Not over water)	0.002	0.002	0.002	0.002

where:

C_2	=	cost of running vehicle, \$0.25/mi
D	=	detour length, mi
A	=	ADT
d	=	duration of detour, d

The duration of the detour is estimated as being an inverse function of ADT, with higher ADT's forcing quicker repair. The assumed repair times are shown in figure 8 and range from 6 months for ADT's greater than or equal to 5,000 to 36 months for ADT's less than 100. This estimated function is based on the higher economic losses and political pressures to act quickly, associated with higher ADT roads. Specific state information may be used to modify this assumed relationship.

The additional time costs are calculated for passenger cars and trucks. AASHTO developed the value of time per passenger hr (\$3.90 for an average trip), value of time for trucks (\$7.00 per hr), and average occupancy of adults per vehicle (1.56). The dollar values were converted to 1991 dollars assuming an 8 percent inflation rate.[14] The resulting time losses are calculated as follows:

$$\text{Time Loss (\$)} = [C_3 O\,(1 - T/100) + C_4\, T/100]\; DAd/S \qquad (13)$$

where:

C_3	=	value of time per adult, \$7.05/hr
O	=	occupancy rate, 1.56 adults
T	=	average daily truck traffic, percent
C_4	=	value of time for truck, \$20.56/hr
S	=	average detour speed, 40 mi/h

Bridge Age

Bridge age (obtained from NBI Item 27 year built) is used as a reality check on the probability of scour failure. The reciprocal of the probability of scour failure is the mean time to scour failure. This is compared to the age of the bridge because age can be used to infer scour risk or foundation adequacy. For example, if the aforementioned methodology yielded a probability of scour failure of 0.1 assuming inadequate foundations (10 yr mean time to scour failure) and the bridge is 80 yr old, one might hypothesize that the foundation assumption is inaccurate even though the state of the foundations is unknown.

Failure probability is assumed to follow a binomial distribution. An upper confidence limit of the 90th percentile was placed as a boundary within which age could be explained by the mean time to scour failure. The 90th percentile mean time to scour figure is calculated for the binomial as follows:

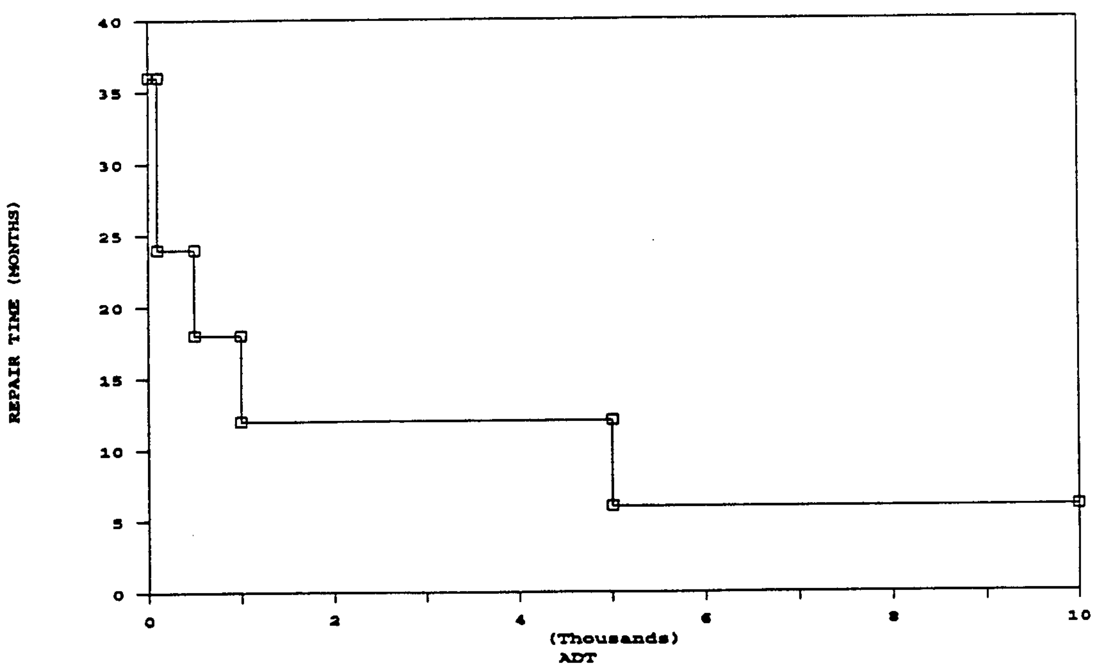

Figure 8. Repair time versus ADT .

$$X_{90} = \frac{\log\ (1-0.90)}{\log\ (1-P)} \tag{14}$$

where:

X_{90} = 90th percentile mean time to scour failure

P = initial probability of scour failure.

If the 90th percentile age, X_{90}, is less than the actual bridge age, then the initially calculated probability of scour failure is revised downward. For example, if the initial probability of scour failure is 0.1 (10-yr mean time to scour failure) and the bridge is 80 yr old:

$$X_{90} = \frac{\log\ (1-0.90)}{\log\ (1-0.10)} = 22 \text{ yr},$$

and since 22 < 80, P, is back calculated assuming X_{90} = 80 using:

$$80 = \frac{\log(1-0.90),}{\log\ (1-P)}$$

to give a revised P = 0.028.

The 90th percentile age versus initially calculated failure probability is shown in figure 9. If the actual age exceeds the maximum expected age for a given failure probability, the probability is revised as previously discussed. The revised probability is then used in determining scour failure risk.

MODEL PERFORMANCE

The model's performance was assessed from a review of NBI data from a Mid-Atlantic seaboard county which listed 78 bridges over water with ADT's ranging from 10 to 58,000. Vulnerability to scour had not been evaluated for 49 of the bridges. The number of bridges with unknown foundations could not be determined.

Where item 113, NBI is not coded as a six, foundation information is presumed to be available and the unknown foundation methodology does not apply. A six indicates that scour calculations have not been made. All bridges within the county have been evaluated regardless of item 113, both to increase the test population and to compare item 113 (where not equal to six) to the vulnerability rating. Item 113 was modified so that all values under six were increased by one, values of six were blanked, and values over six were not altered. This results in a continuous rating from one to nine to compare with the vulnerability rating developed in the methodology. The modified item 113 matches well with the vulnerability rating.

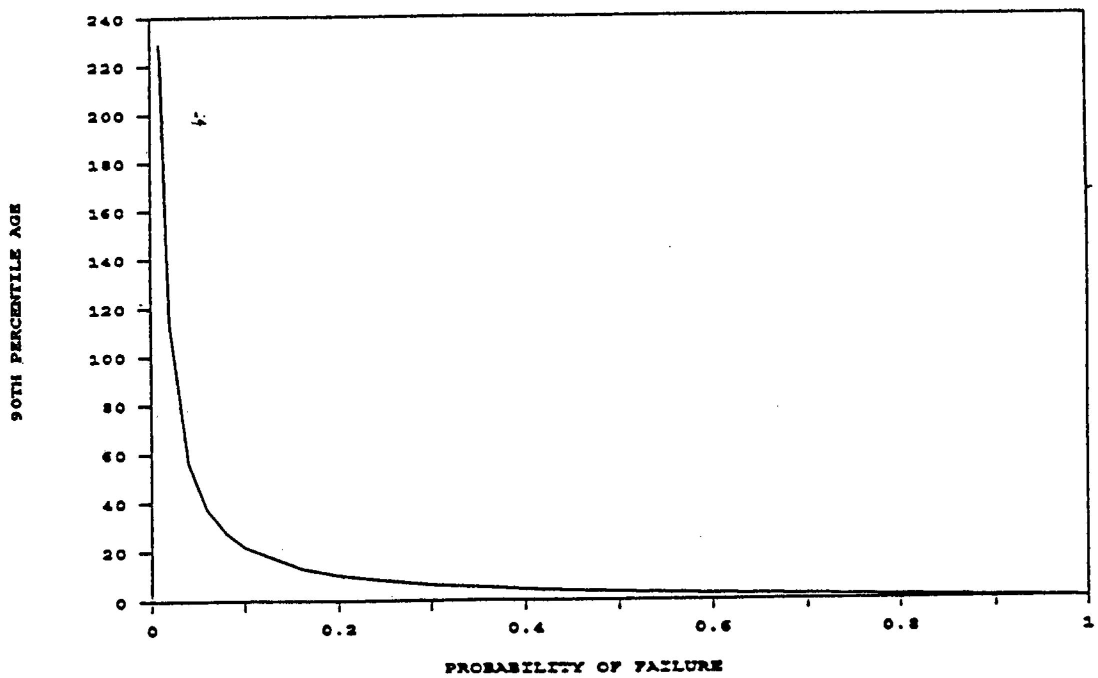

Figure 9. Maximum expected age versus probability of failure.

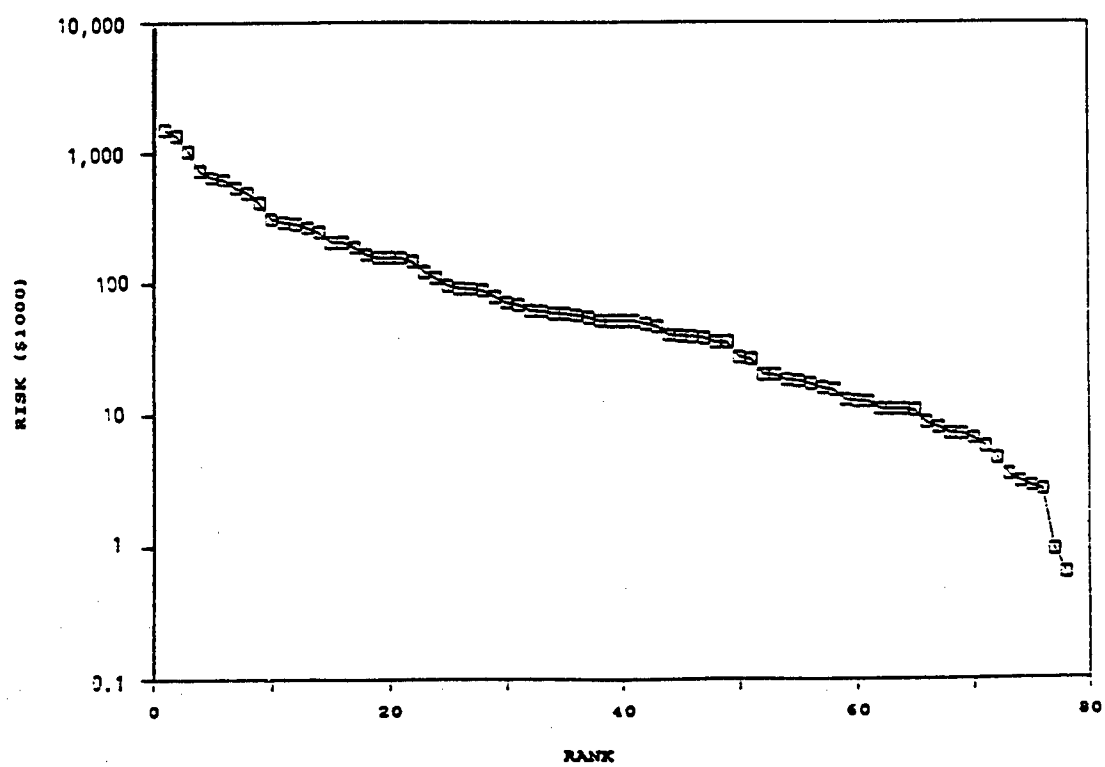

Figure 10. Risk versus rank.

The bridge age is compared to the maximum time to failure and where the age is greater, the probability of failure is revised.

The bridges were then sorted from high to low risk, ranging from $1.5 million to $635. The risk versus rank for this typical county is cumulatively shown on figure 10. Many of the high risk bridges have high ADTs and long detour lengths, both of which influence running costs and time loss costs. The six highest risk bridges have ADT's over 10,000 and 17 of the 24 highest risk bridges have detour lengths of 10 mi or more. Running costs and time loss costs dwarf rebuilding costs for most of the high risk bridges, which is why ADT and detour length are important parameters in this risk based analysis.

To further illustrate the methodology the following example is provided:

Year Built	=	1948	(NBI 27)
A	=	12850	ADT (NBI 29)
T	=	4%	Truck % of ADT (NBI 109)
D	=	34 mi	Detour Length (NBI 19)
L	=	459 ft	Bridge Length (NBI 49)
W	=	25.3 ft	Bridge Width (NBI 52)
Classification	=	6	(NBI 26)
Substructure	=	4	(NBI 60)
Channel Protection	=	6	(NBI 61)
Waterway Adequacy	=	7	(NBI 71)
Scour Vulnerable Bridge	=	6	(NBI 113)
Maximum Span Length	=	150 ft	(NBI 48)
K_1	=	0.67	(NBI 43B, NBI 48)
K_2	=	1	(unknown foundation)

The annual probability of scour failure is determined from the supplied data as follows. Using figure 2, the classification (6) and the waterway adequacy (7) yields a slight (0.02) annual probability of overtopping. Since NBI item 113 is coded 6 (scour calculations not made), the scour vulnerability is determined from figure 7. The substructure condition of 4 and the channel protection code of 6 yield a scour vulnerability of 4. This is defined in table 23 as "Scour vulnerable-action required." The overtopping frequency (0.02) and the scour vulnerability (4) yield a failure probability of 0.11 using table 25.

The bridge age is then used as a reality check on the probability of scour failure. The probability of failure (0.11) indicates a 90th percentile mean time to failure of 20 yr using equation 14. Since the bridge age, 43 yr, is greater than the 90th percentile mean time to failure, the probability of failure is revised downward so that the 90th percentile mean time to scour failure is 43 yr. Using equation 14, the revised probability of scour failure is 0.052.

The repair time is estimated from figure 9 as 6 mo (182.5 d) for an ADT of 12850. This completes the determination of all parameters utilized in equation 3 to determine risk.

Failure costs are then determined as follows:

Rebuilding cost ($)	=	$\$60/ft^2$ WL
	=	60 (25.3) (459)
	=	$696,762
Running Cost ($)	=	$0.25/mi DAd
	=	.25 (34) (12850) (182.5)
	=	$19,933,563
Time Loss ($)	=	[$7.05/hr (1.56 adults) (1 - T/100) + $20.56/hr T/100] DAd/40 mi/h
	=	[7.05 (1.56) (1-4/100) + 20.56 (4/100)] (34) (12850) (182.5)/(40)
	=	$22,685,351

This yields a total failure cost of $43,315,676. Multiplying this cost by the scour failure probability yields a scour failure or heavy damage relative risk of $1,513,181, based on unknown foundations.

SIMPLIFIED FORM

The assessment of model performance for the mid-Atlantic county has indicated that for over 75 percent of the structures listed, the rebuilding cost is less than 15 percent of the total failure cost. Further, structures with highest relative yearly risk cost have the lowest percentage of rebuilding costs, averaging less than 5 percent. This suggests that running and time costs dwarf rebuilding costs and the latter may be omitted in a simplified risk equation without significantly altering the ranking order.

Since running end time costs are interrelated by ADT and detour length and are a function of common constants such as C_1, C_2, C_3, C_4, O, and S, a consequence of failure ranking order may be obtained by simply multiplying ADT by detour length. A comparative but not strictly equivalent risk ranking order for bridges with unknown foundations would therefore be expressed as:

$$\text{Risk Ranking} = K_1 \times P \times ADT \times \text{Detour Length} \qquad (15)$$

For the example developed in the previous section the corresponding Risk Ranking would be obtained as follows:

Revised Probability of Scour Failure	=	0.052
K_1	=	0.67
ADT	=	12,850
Detour Length (mi)	=	34

Risk Ranking = 0.67 x 0.052 x 12,850 x 34 = 15,221

For the mid-Atlantic county analyzed the resulting Risk Ranking order number varies from 1, the lowest relative ranked structure, to 15,221, the highest ranked. The upper and lower 10 percent remain unchanged. Structures with low ADT's and long detour lengths under this simplified format will tend to rank somewhat lower, since time to repair or reconstruct under this simplified format is held constant for all ADT's.

SUMMARY

The method calculates relative annual risks associated with scour failures given NBI data and assuming inadequate foundations. The method was developed to prioritize the gathering of foundation information at sites for which the substructure is unknown. Partial knowledge of foundations reduces the risk accordingly.

Risks are the expected value of losses associated with rebuilding, additional running costs over detours, and lost time. These losses are straightforward to calculate and depend only upon an assumption of how long a failed bridge will take to be repaired. This repair time is assumed to be inversely related to traffic volume. The losses are based on a bridge failure outcome.

Risks weight the economic outcome with a relative failure probability. The methods assume the foundations are poor to begin with (since they are of unknown character, this is the prudent approach). The failure probabilities are calculated as a function of overtopping frequency, substructure, and channel conditions noted by inspectors. The logic and functions are exact at the extreme limits of possibilities and thus will tend to accurately identify higher and lower risks.

For intermediate ranges of possibilities, the method depends upon subjectively determined conditional probabilities and vulnerabilities. The probability calculus is rigorous in the transformations of subjective distributions. A flow chart for this risk methodology is included as figure 11.

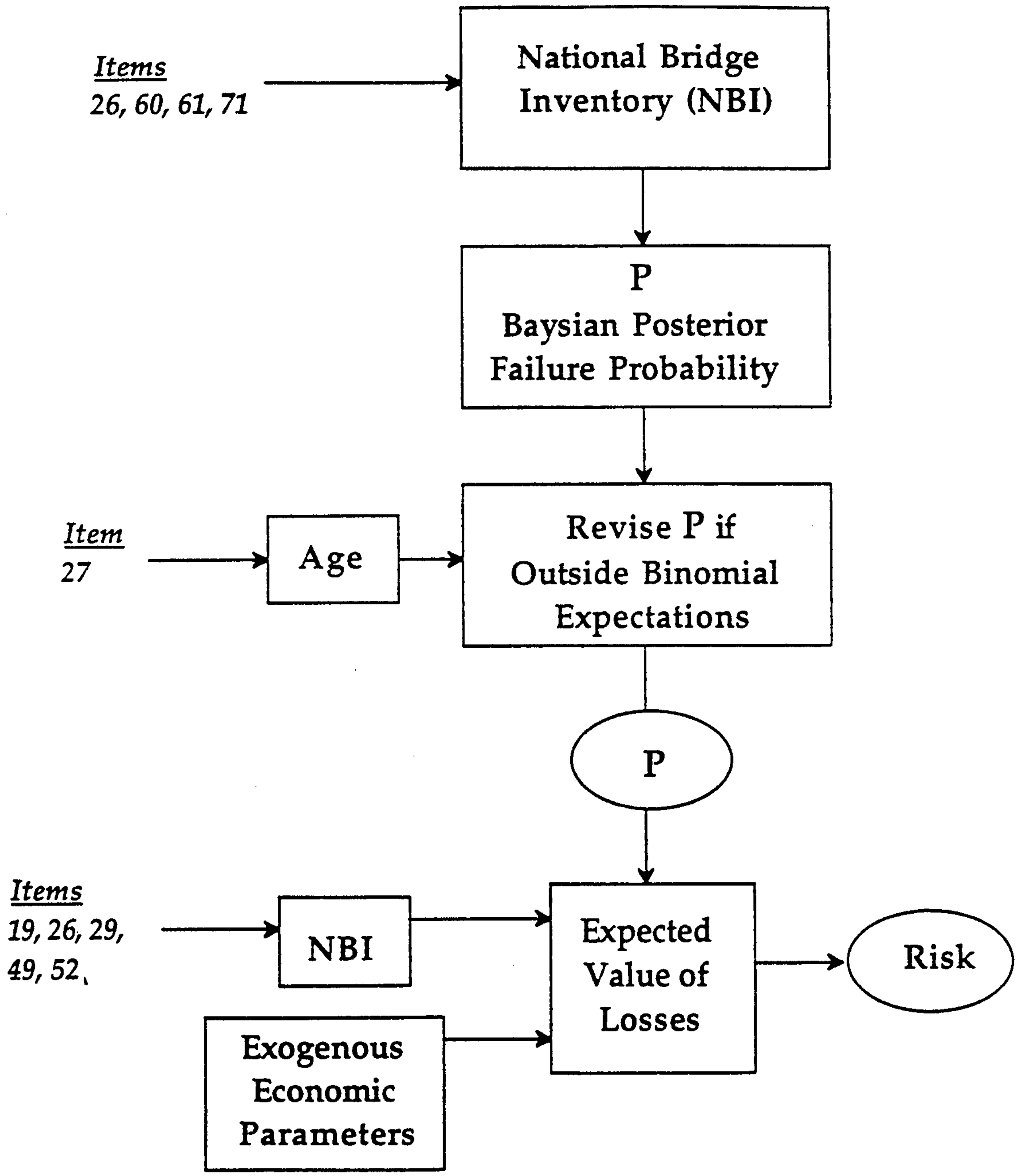

Figure 11. Flowchart for Risk Methodology in Unknown Foundation Prioritization.

Application of the method to an example set of NBI bridges gives reasonable results. The method is sensitive to traffic and detour length. The ranked list of the example set appears reasonable. Users could adapt the approach to their setting and could modify economic parameters to fit local economies.

A simplified method is also presented which capitalizes on the sensitivity to traffic and detour length for the value of losses associated with rebuilding, additional running costs over detours, and lost time. The resulting ranking order is not dollar based and yields rankings which are comparable although not identical.

The ranked risk list will identify those sites that merit immediate investigation. It is very likely that a small amount of additional information, perhaps common knowledge to the responsible parties, will enable the risk estimates to be reduced. Since the method assumes poor foundations, any information to refute the assumption reduces the estimated risk. However, if there is a high initial risk and investigation shows poor foundations, action seems warranted.

A check and balance in the method is the incorporation of bridge age. An old bridge with a high calculated failure probability may be incompatible. Adjustment procedures are included.

Basic assumptions in the method include rational frequencies associated with national USGS flood estimation equations and the fact that shear stress, and hence scour, is depth dependent. The acknowledged weakness is that NBI, until the inclusion of Item 113, is not particularly oriented to items pointing to warnings of flood induced foundation failure; as a result, subjective logic was necessary to interpret the information that is available. Partial knowledge of foundation conditions reduces the risk.

The subjective elements that are addressed in this study are:

1. The vulnerability rating as a tabular function of substructure (item 60) and channel protection (item 61) elements. It is important to note this is a vulnerability to scour failure rating assuming poor foundations to begin with.

2. Marginal failure probabilities related to the vulnerability rating and scaled flow depth; the scaling is related to the maximum distance from the bridge deck to the bottom of the channel.

3. The estimation of repair time as an inverse function of traffic volume.

4. The recognition of the variability in assessment of bridge inspectors, who are:

    - Describing the setting at one point in time.
    - Not particularly focused on flood damage to footings (at least prior to the inclusion of item 113 in the NBI).

Probably not trained hydraulic practitioners although they may be taught what to look for.

The economic parameters of the method that need to be supplied are:

- Value of lost time.
- Occupancy rate.
- Detour speed.
- Running costs.
- Unit bridge rebuilding costs.

The items within the NBI data base that the method uses are:

- Functional class (#26).
- ADT (#29).
- Substructural condition (#60).
- Channel protection (#61).
- Waterway adequacy (#71).
- Year build (#27).
- Width (#52).
- Length (#49).
- Detour length (#19).
- Type of Structure (#43B).
- Length of Maximum Span (#48).

CONCLUSIONS AND RECOMMENDATIONS

1. The NBI data base plus a few economic parameters can be utilized to generate a ranked list of bridges with unknown foundations according to relative estimated risks. The risks are based on the premise that the unknown foundations are generally poor (shallow or susceptible to scour). If some level of data is available, such as type, the list can be re-ranked by utilizing an appropriate additional risk adjustment factor.

2. Subjective determinations are necessary to cope with the lack of stream velocity and depth in the NBI data base. Where State databases contain stream velocity and depth, the degree of subjectivity is reduced.

3. The NBI data base has been expanded to consider scour with the inclusion of item 113. Pending completion of the revised inventory, the risk-based method can prioritize projects for information gathering. At present (1991), 81 percent of structures have not been coded for item 113.

4. The risk-based method can be adapted to the case where the foundations are known to generate rankings that are related to provision of countermeasures. This adaptation can utilize the item 113 information, and foundation risk adjustment factors.

5. It appears, that a significant portion of structures over water, are rural, local, single span, less than 50 ft in length carrying ADT's of less than 100. Based on this risk cost analysis, the dollar based relative risk cost of failure or heavy damage may be less than the cost of determining scour. This is especially true where foundation are unknown and significant dollars need to be expended on their determination.

6. It is recommended that the method be Beta tested with the aid of Federal officials or a state agency, with revisions as required.

7. Implementation of this method holds promise for rationally directing scarce information gathering and analysis manpower. It can be applied to both known and unknown foundation situations.

CHAPTER 4. PRACTICAL AND FEASIBLE GUIDE

GENERAL

The objective of this portion of the study is to define and describe operational methods of determining unknown subsurface bridge foundations keeping in mind that unknown foundations are being defined by states as either:

- A foundation whose type is known but its bottom elevation unknown.
- A foundation whose type and elevation is unknown.

Further, the foundation units to be investigated may be either:

- Piers on land or water.
- Abutments.

The methodologies presently available can be conveniently grouped as:

- Direct methods.
- Deductive methods.
- Emerging methods.

Each of these grouped methodologies have costs, range of applicability, and accuracy uniquely associated with each group. For multiple span bridges it is possible that a number of methods need to be utilized.

For some structures, the costs associated with determining foundation type or depth may outweigh the risk cost associated with a potential failure as developed in chapter 3.

Further, it is believed based on discussions with State agencies that information with respect to foundations, not presently contained in State databases, may exist in local agency or district files. This search should be made prior to implementation of any of the methodologies subsequently described.

DIRECT METHODS

The direct methods consist of locating structure elements and obtaining measurements of visible elements and piers.

Subsurface elements on land or water are located by means of probes, hand augers, power augers, rotary drilling methods, or digging of adjacent test pits to locate the top of footing. The thickness of footing may be either estimated based on local construction practice or by rotary drilling methods thru the footing.

Drilling methods and procedures are outlined in the 1988 AASHTO, Manual of Subsurface Investigations. These methods will be especially applicable where the type is known or there is a strong presumption that the substructure units are on spread type foundations. Adjacent test pits can determine the existence of piles in substructure units as well as their size, spacing, and condition.

It is estimated that a significant portion of structures with unknown foundation can be economically investigated, using conventional drilling methods. The cost of such investigations for one span bridges (50 percent of total) should not exceed the costs associated with 1 rig day per structure, plus mobilization. On a commercial purchase basis this costs should not exceed $1,500 (1991). Since many Transportation Agencies have their own drilling equipment and maintenance divisions with backhoes, the costs basis may be lower.

DEDUCTIVE METHODS

These methods are especially applicable where the type is known or there is a strong presumption that the substructure units are supported on deep foundations (piles, drilled shafts, etc).

They rely on geotechnical analyses and investigations to deduce the most likely founding elevation from the nature and density of the subsurface strata encountered and sampled.

These methods fall in the following categories:

Boring with SPT sampling

Borings are taken alongside the foundation to determine the depth of possible support strata. Where access to the foundation is difficult, rigs may be placed on the bridge deck and proceed to core thru the deck and thru the footing, or within the abutment area. Sampling using the Standard Penetration Method (SPT) ASTM D-1586, or Cone Penetrometers, should define the density of each encountered strata and therefore lead to a deduction as to tip elevation of the piles, as most piles are driven to practical refusal in very dense layers. The density of the strata as deduced by SPT blows is generally indexed as shown on table 26.

Table 26. Penetration and soil properties on basis of the Standard Penetration Test.

Sands (Fairly reliable)		Clays (Rather unreliable)	
Numbers of blows per foot N	Relative Density	Number of blows per foot N	Relative Density
0-4	loose	2-4	soft
10-30	medium	4-8	medium
30-50	dense	8-15	stiff
over 50	very dense	15-30	very stiff

Although determination of likely tip elevations based on SPT data is beyond the scope of this guide, it can be postulated that refusal is likely to have occurred after 5 to 15 ft penetration in deposits classified as dense to very dense or very stiff to hard.

Seismic Refraction Methods[5]

Seismic refraction is based on the principle that elastic waves are refracted upon entering a material of different elasticity.

The method consist of producing a shock wave in the soil or water by exploding a small charge or by a hammer blow on a plate placed on the surface of the ground and measuring the elapsed time between initiation of the wave and "first arrival times" for the wave to reach a geophone. The velocity of the wave can then be determined from the distance it traveled and the elapsed time. The data obtained is plotted in the form of distance against elapsed time from which strata velocity and depth to strata can be obtained.

The velocity of each strata is a rough indicator is its composition and density. Typical seismic velocities of various materials are given in table 27.

Table 27. Typical seismic velocities.

Material	Velocity (ft/sec)
Alluvium	1,640- 6,600
Clay	3,000- 9,200
Glacial Moraine	2,500- 5,000
Glacial Till	5,600- 7,400
Gravel	1,500- 3,000
Sand	4,600- 8,400
Cemented Sand	2,800- 3,200
Loose Sand	5,900
Loose rock talus	1,250- 2,500
Fresh water	4,700- 5,500
Soft Shale	2,600- 8,000
Hard Shale	9,000-15,400
Sandstone	4,600-14,000
Soft Limestone	5,600-14,000
Hard Limestone	16,000-20,000
Decomposed Granite	1,500- 2,200
Sound Granite	13,000-20,000
Weathered Basalt	9,000-14,000
Sound Basalt	18,300-21,100

Because of the obvious overlap of seismic velocities, seismic refraction surveys are not good indicators by themselves of the density of various strata and require borings for indexing purposes. On water, marine seismic systems that are towed can be used to map stratigraphically geological formations, using impulse sound sources. Resolutions achieved with high frequency energy sources may be on the order of 1 ft, with penetrations of up to 100 ft, in depths of water of up to 100 ft.

The cost of seismic refraction surveys per d are on the same order of magnitude as costs per rig day of performing borings. However up to 2000 linear ft/d can be successfully mapped on land, and therefore these methods may be considered for determining stratigraphy under piers for long multispan structures, where the water opening is small or usually dry and at least one borings has been made at one unit for indexing purposes.

Resistivity Methods[5]

The basis of resistivity methods is Ohm's Law. Different materials offer different resistance to passage of an electrical current. By determining the vertical and lateral variations of soil layers, within severe limitations, it is possible to infer the lateral extent and stratification of soil deposits.

Surveys are conducted in accordance with ASTM G-57. Two types of resistivity surveys are used for subsurface exploration, electric profiling and electric sounding. Profiling provides information concerning lateral variations and sounding information on the variation of subsurface material with depth. The latter is accomplished by maintaining the center of the electrode spread at a given location and taking a series of resistivity readings as the electrode spacing is increased. Several methods have been developed to interpret results of electric sounding surveys and determine the depth to layers.

Although there are some instances of successful use of this method, many factors affect field measurements and the data can often be misleading or difficult to interpret. Among the factors affecting interpretation are the broad range of resistivity values as shown on table 28 for a given material, the overlap of values for different materials, the moisture content, degree of saturation, near surface irregularities, stray current potentials, and the level of soluble salts in the soil and rock.

Table 28. Representative values of resistivity for dry soil.

Material	Resistivity (Ohm-cm)
Clay	300 - 5,000
Sandy Clays	1,000 - 10,000
Sand	1,000 - 17,000
Gravel	5,000 - 20,000
Weathered Rock	3,000 - 200,000
Sound Rock	5,000 - 4,000,000

It should be noted that moisture content significantly affects resistivity. For a given soil, resistivity decreases rapidly with increasing moisture content until the saturation is reached after which resistivity remains essentially constant.

The severe limitation outlined restricts the usefulness of this method to areas where the ground water is high and rock is relatively near the surface. In general the method is not as reliable as seismic exploration but might be of limited interest due to its low equipment and manpower cost. It is estimated that one technician with portable equipment can easily perform the required surveys in less than 2 hr at each foundation location.

EMERGING METHODS

These methods are applicable to a wide range of situations ranging from totally unknown foundations to the direct determination of depth of piling for units supported on visible pile bents or piers. The methods are derived either from well developed nondestructive (NDT) techniques developed to assess the quality of piles, drilled shafts and slurry trenches after installation, adaptations of ground penetration radar (GPR) techniques, or acoustical emission techniques.

The applicable NDT techniques are based on:

- Low strain methods such as Pulse Echo transmission.
- High strain methods such as Transient Dynamic Response.

It should be noted that a current FHWA research project is in the process of evaluating for NDT techniques, the relative advantages/disadvantages, limitations and costs for quality control procedures when the shaft top is exposed, and available for testing.

The adaption of these techniques to more complex units such as footing-pile, bent-pile, or abutment-wall-pile combinations has not been consistently demonstrated, although claims of reasonable success has been made by specialists in this area of NDT, working for a number of highway departments. The principles, limitations, and costs for all current commercially available low strain methods are outlined as follows:

Sonic Echo or Pulse Echo Test[6]

This technique is based on a time analysis of stress wave propagation.

The earliest of all low-strain tests available commercially, this method was developed concurrently by several groups. It is known variously as the Echo test or TNO method, (TNO, Holland), the Impact-Echo test, (CEBTP, France), and the PIT (Pile Integrity Test, Pile Dynamics Inc, USA). It is offered by a number of other companies under license to one of these groups, where it is also sometimes known as the Sonic-Echo method.

The Pulse Echo method, as its name implies, uses a small impact delivered at the head of the shaft and monitors the time taken for the stress wave generated by the impact to travel down the shaft and be reflected back ('echo') to the head of the shaft. Typically the impact is from a small sledge hammer, which contains an electronic device to record the moment of the impact. The reflected wave is detected by an accelerometer, and both instruments are connected to an oscilloscope or a digital data acquisition device which records the data on a simple time-base.

If the length of the shaft is known, and the transmission time of the stress wave is measured, then its velocity can be calculated. Conversely, if the velocity is known, then the length can be calculated. Since the velocity of the stress wave is primarily a function of the elastic modulus and density of the concrete, the calculated velocity can provide information on concrete quality as well.

Where the stress wave has traveled down and back up the shaft, these calculations are based on the formula:

$$Vc = \frac{2l}{t} \tag{16}$$

Where:

Vc = Stress wave velocity in concrete
l = shaft length
t = Transit time of stress wave

Empirical data has shown that a typical range of values for Vc can be assumed where 3800 m/s to 4000 m/s would be indicative of good quality concrete, with a crushing strength in the order of 30-35 N/mm^2. The actual correlation will vary according to aggregate type and mix, and these figures can be used only as a broad guide to concrete quality.

The energy imparted to the shaft by the impact is relatively small, and the damping effect of the soils around the shaft will progressively dissipate that energy as the stress wave travels down and up the shaft as shown on figure 12.

Depending on the stiffness of the lateral soils, a limiting length/diameter ratio exists, beyond which all the wave energy is dissipated, and no response is detected at the head of the shaft. In this situation, the only information that can be derived is that there are no significant defects in the upper portion of the shaft, since any such defect closer to the head than the critical L/D ratio would reflect part of the wave. This limiting L/D ratio will vary according to the lateral soils, but a typical value in medium stiff clay is 30 : 1.

The polarity of the reflected wave will depend on the relative support conditions at the point from which it is reflected. It is possible, for example, for a bulb, or oversize cross-section, in a layer of loose soils, to reflect a waveform similar in polarity to that which would be reflected by a reduction in shaft section in a layer of stiff soils.

These limitations make the method unsuitable for shafts with high length/diameter ratios and shafts where there is no control of maximum cross section.

The Pulse Echo method was developed for shafts where the maximum cross section is known and controlled, such as pre-cast driven piles, or permanently cased drilled shafts. It is particularly effective in softer soils.

In these applications the method can measure shaft length or footing thickness and confirm integrity, identify discontinuities, or inclusions, and provide a comparative guide to concrete quality.

Transient Dynamic Response Test[6]

This technique is a frequency based analysis of stress wave propagation, where a blow on the head of the shaft from a small sledge hammer with a special impact face generates a stress wave with a wide frequency content which emulates the effect of a vibrator. The input force is measured by a solid-state load cell or force transducer built into the hammer head.

The response of the shaft is monitored by a geophone velocity transducer, and the signals from the two instruments are recorded by a digital data acquisition device. Unlike the Echo type test, which does not measure force, the force and velocity signals from the TDR test are processed by computer, using the Fast Fourier Transforms (FFT) algorithm, to convert the data into the frequency domain. Velocity is then divided by force to provide the unit response, or transfer function, which is displayed as a graph of mechanical admittance for the shaft as shown on figure 13.

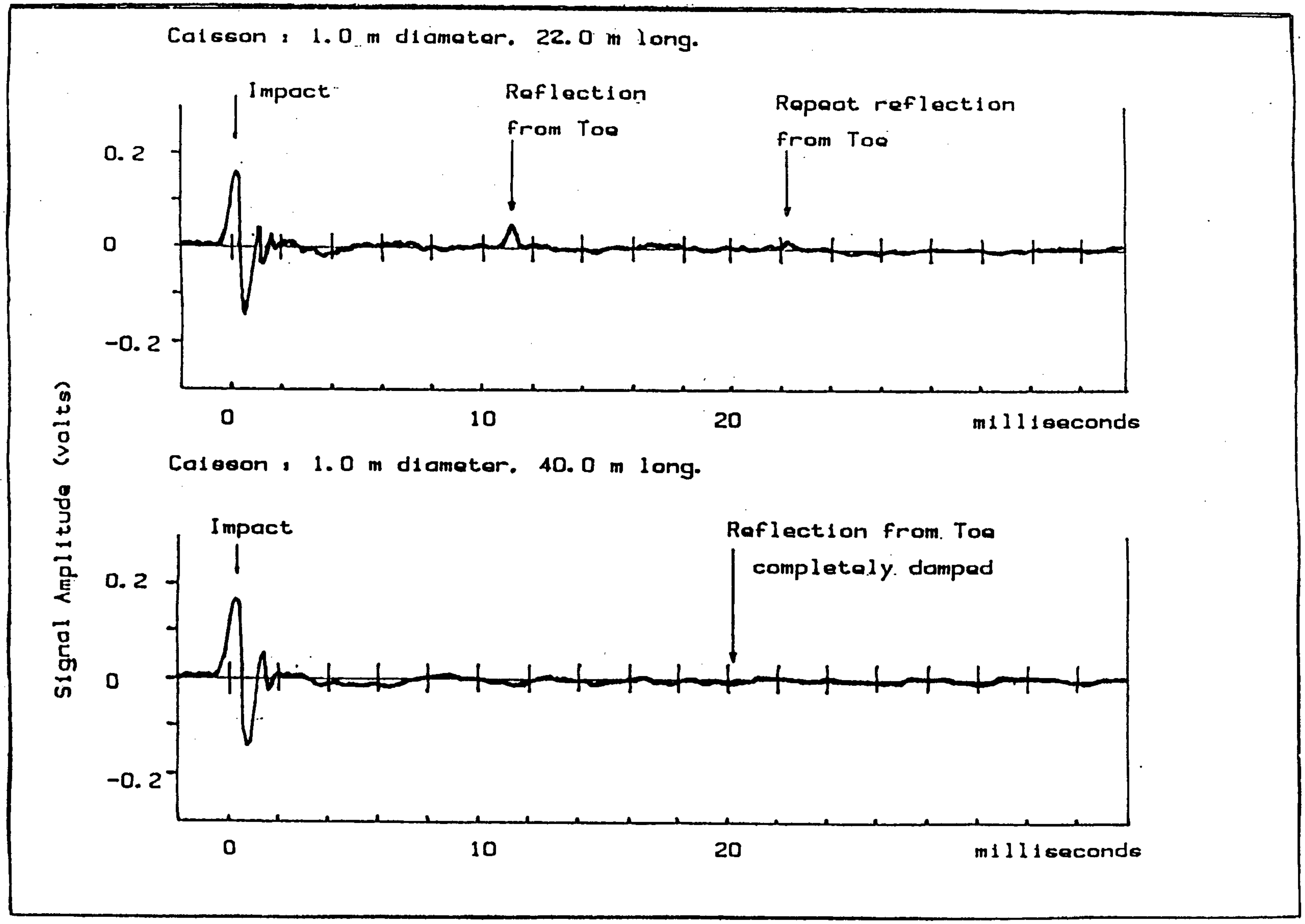

Figure 12. Reproduction of typical early impact-echo test result.

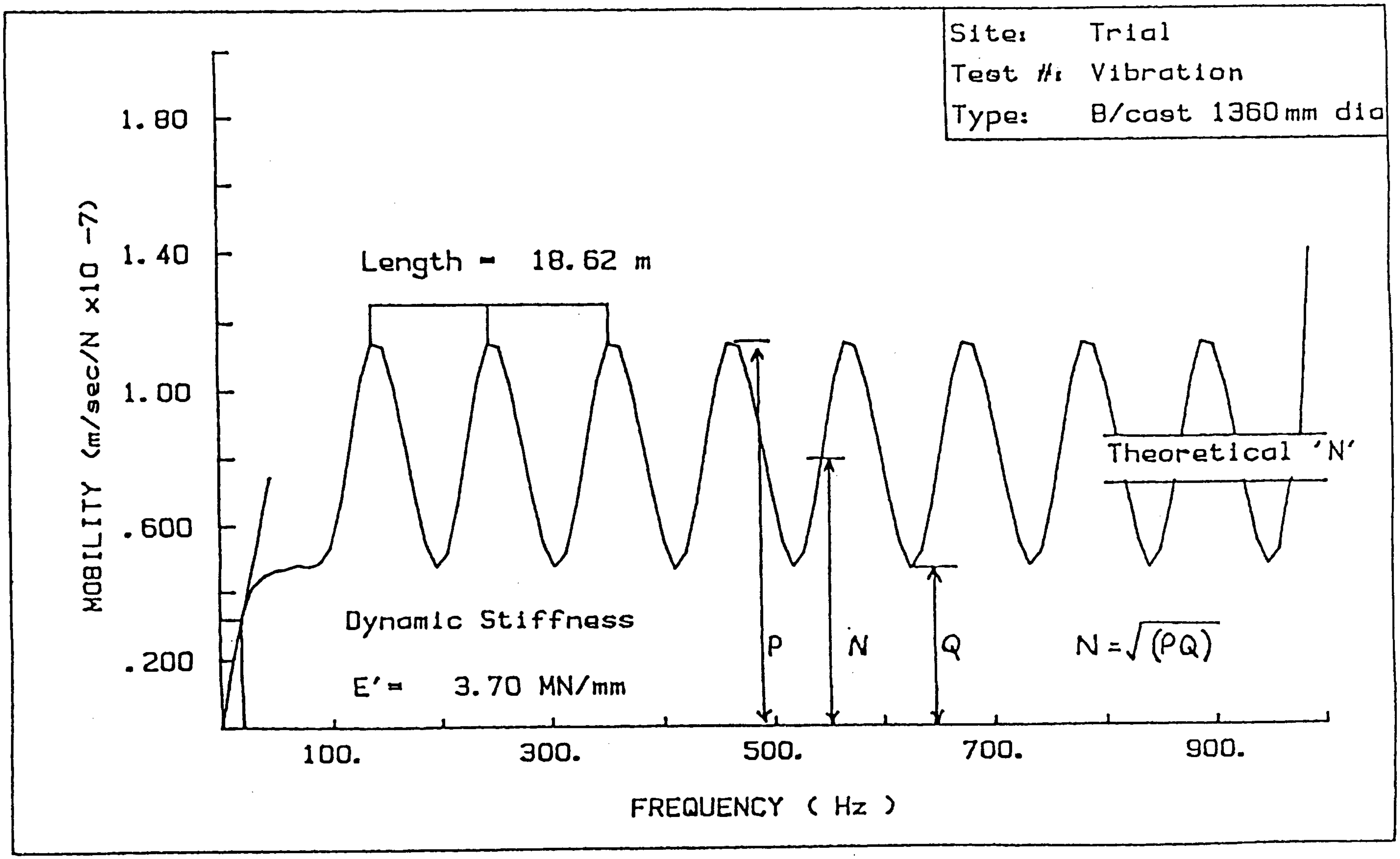

Figure 13. Typical harmonic response of caisson.

At low frequencies lack of inertial effects causes the pile/soil complex to behave as a spring, and this is shown as a linear increase in amplitude against frequency from zero to the onset of resonance. The inverse of the slope of this portion of the graph can be used to calculate the dynamic stiffness of the shaft head, as follows:

$$E' = \frac{2\pi}{(V_0/F_0)_m} f_m \qquad (17)$$

Where : E' = Dynamic Stiffness

f_m = Frequency

$(V_0/F_0)_m$ = Mobility (coordinates of point M)

The dynamic stiffness is a property of the shaft/soil complex, and can therefore be used to assess a number of shafts on a comparative basis, either to establish uniformity or to select a representative shaft for full-scale load testing.

The higher frequency portion of the admittance curve is where the shaft response has gone into resonance. The frequencies of these resonances are a function of the shaft length, and their relative amplitude a function of the lateral soil damping effects. The mean amplitude of the resonant portion of the curve is a function of the cross-sectional area of the shaft, the density of the concrete, and the propagation velocity of the stress wave (Vc).

As with the Echo type tests, where the length of the shaft is known, a shorter length measurement will indicate the presence of an anomaly. However, with the additional information available from the TDR method, such as cross section and dynamic stiffness, it is possible to differentiate between an increase and a reduction in cross section, even in complex soils.

In the frequency domain, the phase of resonant frequencies is a more sensitive indicator of relative support conditions than the polarity of the time-based signal used for the Impact-Echo test, and it is often possible to assess the condition of the interface between shaft base and bedrock from TDR test results.

Since a relatively small amount of energy is generated by the hammer impact, soil damping effects limit the depth from which useful information may be obtained. However, even where there is no measurable response detected from the base of the shaft, the dynamic stiffness value obtained is still an important parameter for shaft assessment.

The TDR method was developed specifically to meet the shortcomings of the Echo methods on shafts where cross section was likely to be variable. It is therefore suited to open drilled shafts, temporarily-cased driven cast-in-place shafts, and augured cast-in-place (continuous-flight auger) shafts.

The above methods were originally developed as quality control tests for concrete piles where the top of the pile was exposed and available for testing. For existing pile bent structures, footings over piles, pier-footing-pile combination etc., adaptations must be made in order to introduce the signal and process and interpret the return signals.

These adaptations include:

- Notching a horizontal surface in a visible pile in order introduce the signal.
- Constructing a stiff bracket attached to the pile to introduce the signal.
- Introducing a shear wave signal in addition to the normal longitudinal signal.
- Introducing the signal on top of the footing or pier directly over the pile.

It should be noted that these adaptations introduce additional potential limitations such as:

- Reduction of effective length/diameter ratio to perhaps 20.
- The possibility of no signal return if a void exists between top of pile and footing.
- Uncertain signal return for wood piles and steel piles.
- For steel piles, the cross-sectional area may be too small to generate a return signal without access to the top of the pile.

The outlined limitations would suggest that these methods would require additional development, mainly in signal processing and enhancement to increase reliability. The introduction of recognition or neural networks to aid in signal processing would be an important advance.

The cost of NDT surveys per d are on the same order of magnitude for all methods, $1,200-2,000 per day.

Parallel Seismic Method[6]

This technique based on acoustic wave propagation, is a through transmission technique. The methods outlined in the previous sections, for maximum efficiency, depend on access to the head of the shaft. The parallel seismic method was developed specifically for the evaluation of the foundations on older. existing structures, when direct access to the shaft head is no longer possible without structural damage.

For implementation a small diameter access borehole is drilled parallel and close to the foundation to be tested. The borehole must extend beyond the known or estimated depth of the foundation, and is normally lined with a plastic tube to retain water as an acoustic couplant. A receiving probe is placed in the tube at the top, and the structure is struck, close to the head of the foundation, with a hammer containing a device which registers the moment of impact.

Signals from both instruments are recorded on a data acquisition unit which records the time taken for the sound of the impact to travel thought the foundation and adjacent soil to the receiving probe. The probe is then lowered in uniform increments, and the process repeated at each stage, with the impact being in the same place each time. The recorded data is then plotted out with each measurement printed immediately below the previous one, providing a vertical profile of the transit time of the soundwave from the point of impact to each position down the access tube.

The velocity of the soundwave through soil will be lower than its velocity through concrete or steel. If the access tube is assumed to be placed reasonably parallel to the foundation, the effect of the soil between on the transit time of the signal will be fairly constant. However, transit time will increase proportional to the increase in foundation depth. A line drawn on the profile graph to link the first arrival of each successive signal will have a uniform slope as shown on figure 14.

When the receiving probe has passed beyond the foundation base, the transit time of the acoustic signal will be extended by the lower velocity of the additional intervening soil, and the lines linking signal arrival points on the graph will show a distinct deflection at the level of the foundation base.

Similarly, any significant discontinuity or inclusion in the foundation will force the signal to detour around it, increasing the path length and transit time.

The acoustic transmission properties of soil vary with density, moisture content, and cohesiveness. This variability will affect the lateral range of the Parallel Seismic method, and typically the access tube must be within 3 ft of the foundation to be examined.

The method does have depth limitations, but these are not so easy to define as those of the TDR and Echo methods. Both the amplitude and dissipation of the acoustic signal will depend on the soil conditions and the material of the foundation. The shape of the foundation and the proximity of the access tube will also affect the maximum depth of penetration. Each case should be assessed by an expert in the method to determine its feasibility.

If the foundation has a variable cross section, such as a stepped drilled shaft, it may be difficult to differentiate between defect and a deliberate change in cross section.

The Parallel Seismic method can be used on most concrete and steel foundation types, including steel sheet piles and 'H' piles, where the head of the foundation is not directly accessible.

The method has been successfully used on large masonry and mass concrete foundations, for dams, and on heavy machine bases. It has also been used with

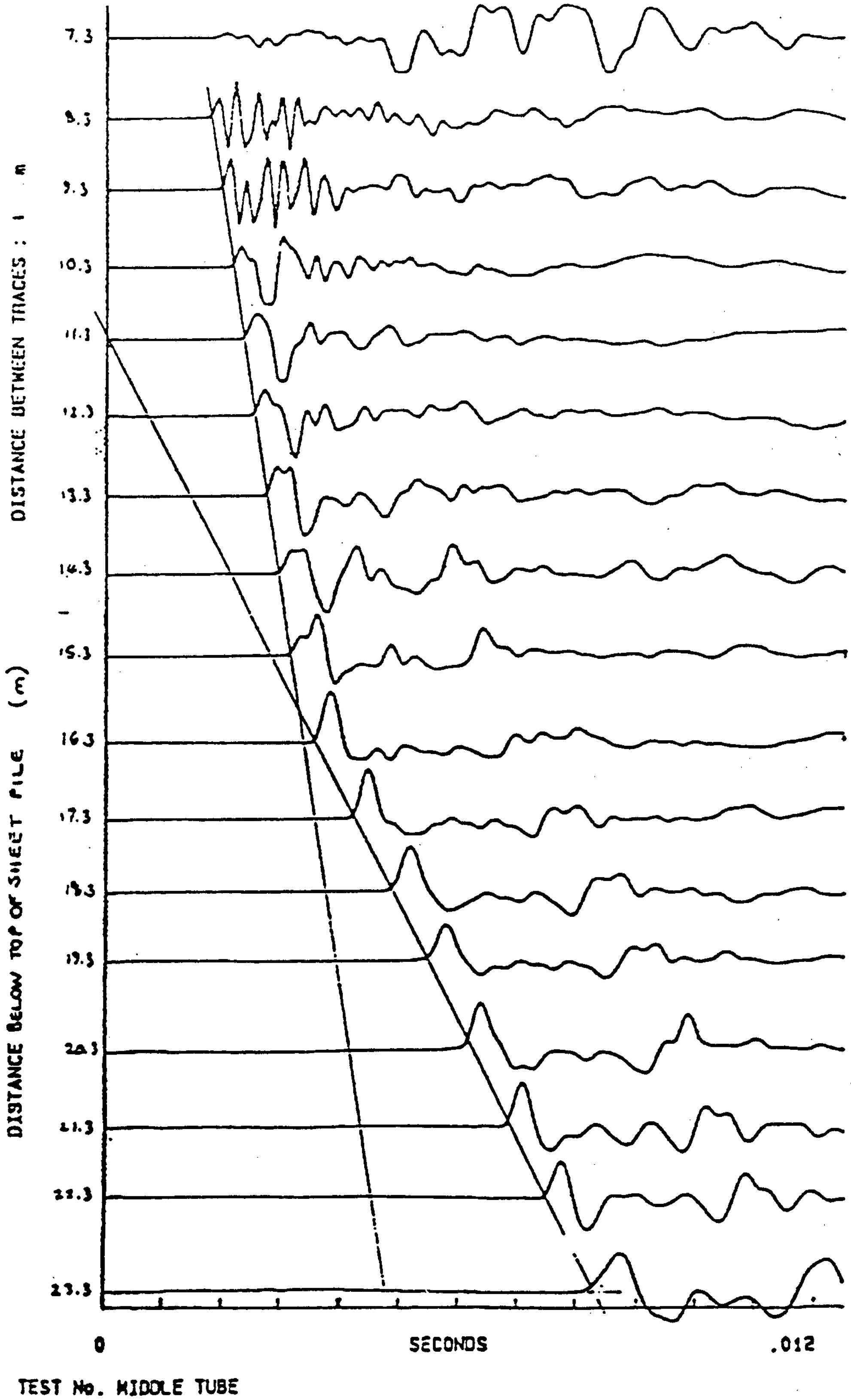

Figure 14. Parallel seismic trace.

some success on certain types of timber foundation and for the assessment of cracking in large dams.

Pulse Echo in Borehole[7]

The technique is similar in concept to the parallel seismic method previously outlined, except that both the impactor and receiver are mounted in a single unit and lowered in the borehole at successive predetermined depths. The impactor generates a single acoustic wave which is reflected when it hits an obstruction and is captured in the receiving head. Processing and recognition of the signal locates the obstruction (pile, etc.) at that elevation.

The method has limitations based on the type of subsurface strata encountered, because of the dissipation of the acoustical signal generated. Best signal reception is obtained in sand profiles, with accuracy decreasing with increasing silt or clay content. Signal recognition, processing, and enhancement are the key in successful implementation of this method. A schematic representation of the set up and trace is shown on figure 15.

Impulse Radar[7,8]

Probing subsurface conditions by impulse radar, referred to as ground penetration radar (GPR), electromagnetic subsurface profiling (ESP), and subsurface interface radar has been in use since 1970 for continuous profiling of strata between borings, determination of bedrock surfaces, and the location of buried objects. It can be adapted to determine the thickness of concrete footings, if access is available, and to possibly determine the existence of piles below a foundation.

Ground penetrating radar can be used on land or in relatively shallow (less than 20 ft) water to obtain high resolution, continuous subsurface profiles. The method is based on transmitting short, 80 to 1000 MHz (megahertz) electromagnetic pulses in to the subsurface by a transmitter located in the antenna and receiving and processing the energy reflected from subsurface interfaces.

The impulse radar system radiates repetitive short time duration electromagnetic pulses from a broad band antenna placed in close proximity with and electromagnetically coupled to the surface. The equipment functions as an echo sounding system using radar pulses of only a few nanoseconds which are emitted while it is being towed in a predetermined pattern and are recorded graphically or on magnetic tapes for later processing. The profile data is displayed in black and white or color line scans similar to those used in marine bottom or sub-bottom profiling or in wiggle plot formats commonly used by geophysicists. Analysis of data requires calibration for accurate depth determination by measurement of echo travel time of a pulse from a conspicuous subsurface interface or other target or when two antennas are used by measurement of the effective propagation velocity.

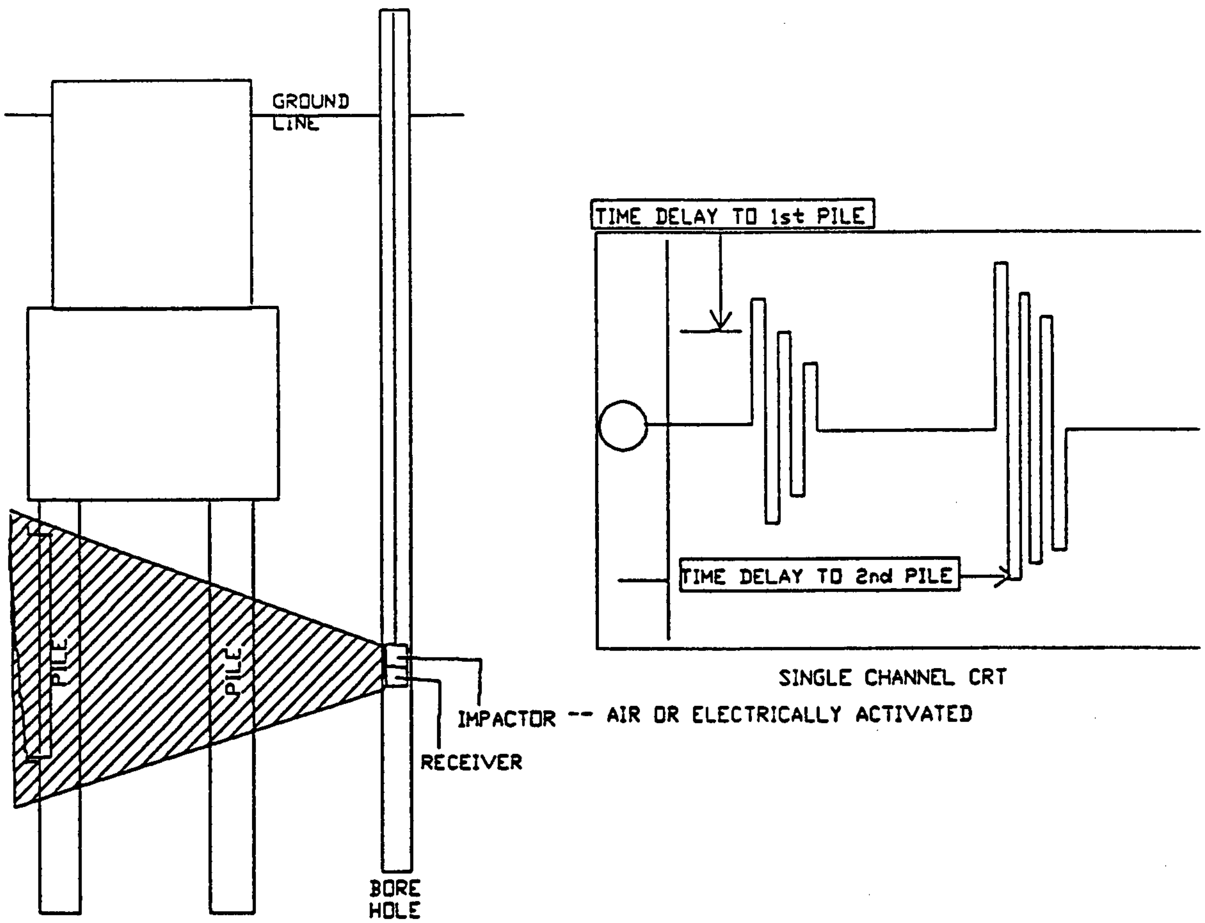

Figure 15. Pulse echo system with borehole.

GPR seldom penetrates more than 100 ft into the subsurface, and in highly conductive materials, it may penetrate only a few ft. The resolution is dependent on operating frequency. The penetration depth depends on the electrical properties of the material through which the pulse is propagated. High conductivity (low resistivity) materials severely attenuates radar signals. Radar is ineffective where sediments are saturated or overlain by salt water or other conductive fluids. Fresh water also attenuates the signal and generally limits the use of radar to sites overlain by less than 20 ft of water.

Antennas with high frequencies of say 300 to 1000 MHz produce greater resolution of detail over a shallow depth, whereas antennas with low frequency of say 80 to 120 MHz provide greater penetration but with less resolution.

Under the best conditions stratigraphic changes may be well delineated, but the nature and density of the material can only be estimated from the relative dielectric permittivity which is a dimensionless ratio of the permittivity (ability to store an electric charge) of a given medium to that of free space. Approximate constants are tabulated on table 29.

Table 29. Approximate dielectric permittivity of typical materials.

Material	Relative Dielectric Permittivity
Air	1
Fresh Water	81
Sea Water	81
Sand (Dry)	4-6
Sand (Saturated)	30
Silt (Saturated)	10
Clay (Saturated)	8-12
Permafrost	4-8
Granite	5
Limestone	7-9
Concrete	6-7
Peat (Wet)	50-60
Peat (Dry)	5-8
Coal	4-5
Dolomite	6-8

The sending unit can be oriented at an angle to the foundation in certain cases and beamed to look beneath a foundation as schematically shown on figure 16.[7]

The cost of GPR surveys per day are on the some order of magnitude as costs per day of performing seismic surveys. It should be considered to determine the existence of piled foundations and to delineate stratigraphy under piers for long multi-span structures over relatively shallow water and where at least one boring has been made at one unit for indexing purposes. The additional advantage of GPR surveys is that it holds some promise in identifying old scour surfaces.[8]

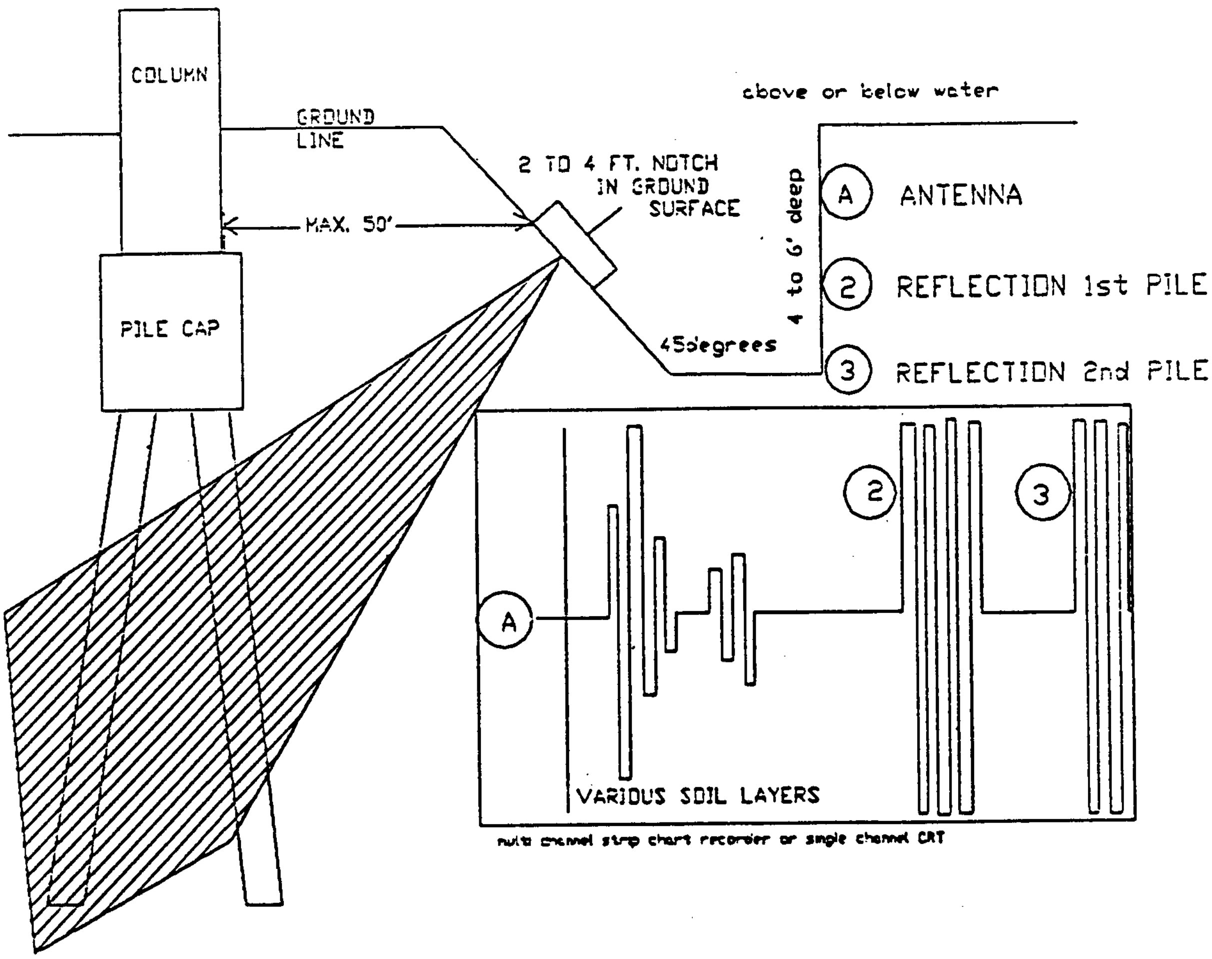

Figure 16. Impulse radar with angle notch in ground.

SUMMARY

The outlined methods range from conventional "dig and see", to deductive methods based on geotechnical analyses, to emerging technologies which hold promise in directly determining the type and depth of foundations, at relatively comparable costs.

Table 30 summarizes the described methods, their application, limitations, and approximate cost.

Since the majority of structures classified as unknown (with no information whatsoever) are short, one span structures, the "dig and see" methods performed by state maintenance forces may be the most expeditious and cost effective. Where piles are the visible means of support or known from file information, deductive methods based on borings with SPT sampling may again prove most cost effective as many DOT's maintain their own drilling rigs and crews for design exploratory work. These methods will not ascertain the length of piles but may determine that piles are longer than the anticipated maximum scour length. The additional advantage of boring surveys is that the nature of the subsoil strata can be definitively identified and may have a bearing on scour depth evaluations.

For multiple span structures a combination of one boring and a NDT type of survey may be most appropriate. The actual type of NDT testing chosen to augment the boring cannot be predetermined, as it is a site specific selection which is dependent on access, foundation configuration, nature of subsurface soils etc. The method should be chosen by a specialist who is capable of performing all of the identified methods. The most promising, as well as the most widely used at present, appears to be a low strain echo method (pulse echo, etc.) whether applied directly on foundation units such as piles, pile bents, piers, or in a borehole. It should be noted that the state of the art for NDT methods is still evolving. The best technology appears to be largely proprietary especially in the software for signal enhancement and processing.

In the future it appears that major improvements for low strain methods or high strain methods, generally in the public domain, should concentrate on processing and filtering the signal, and then via neural networks and curve fitting procedure, automate signal recognition and analysis. At present, interpretation is still somewhat subjective based on the interpreter's judgement and experience. The introduction of neural network based systems should enhance reliability, especially if calibrated to a significant number of known situations.

Table 30. Summary of methods of determining unknown foundations.

Method	Principle	Application	Limitations	Approximate Cost
1. Direct with probes, augers, core drills, etc.	Uses exploratory drilling or probes for location of subsurface elements. Cores footings to establish footing thickness	Spread foundations on land	Accessibility of foundations elements	$1500 per one span structure
2. Direct with adjacent test pits	Exposes subsurface elements for visual inspection and direct measurements	Spread or piled foundations on land	Accessibility. Does not determine length of piles	$1000 per one span structure
3. Deductive with SPT sampling	Infers foundation elevations of subsurface units from soil density correlated to blow counts	Spread or piled foundations	Accessibility. Provides approximate elevation of sub-surface units. Requires analysis of developed field data	$1500 per one span structure
4. Deductive with seismic refraction surveys	Infers foundation elevations of subsurface units from developed subsurface stratigraphy based on seismic velocities of soils	Spread or piled foundations. Multi span bridges	Not practical in deep water. Provides approximate elevation of bearing strata. Requires geotechnical analysis	$2000 per structure
5. Deductive with resistivity surveys	Infers foundation elevations of subsurface units from stratigraphy based on resistivity of soils	Spread or piled foundations on land	Not practical in saturated soils. Provides approximate elevation of bearing strata	$500 per one span structure

Table 30. Summary of methods of determining unknown foundations (Continued).

Method	Principle	Application	Limitations	Approximate Cost
6. NDT using Pulse echo	Measures propogation time of longitudinal waves in concrete	Spread or piled foundations. Particularly adapted for multi span exposed pile bent structures	Unsuitable for drilled shafts with high length diameter ratios. Requires access to foundation units and preparation of surface to introduce signal. Signal return for wood or steel piles may be uncertain	$2000 per day plus mobilization
7. NDT using Transient dynamic response	Measures dymanic response in the frequency domain	Spread or piled foundations. Particularly adapted for multiple span pile bent structures or piers on piles	Same as 6.	Same as 6
8. NDT using Parallel seismic or Pulse echo in borehole	Measures acoustic wave propogation from a probe placed in a borehole adjacent to the foundation unit or with impactor and receiver in borehole	Spread or piled foundations	Borehole with acoustic probe must be drilled within a few feet horizontally of foundation. Best in cohesionless soils	$2000 - $3000 per one span structure plus mobilization
9. NDT using Impulse radar	Measures electromagnetic pulses radiated in soil and recaptured to graphically indicate obstructions	Spread or piled foundations	Not effective in salt water or saturated soils in delineating stratigraphy. May identify location of foundation elements when angled	$2000 per one span structure plus mobilization

REFERENCES

1. FHWA (1991) Hydraulic Engineering Circular No. 18, Scour at Bridges. Federal Highway Administration. Washington, D.C.

2. Chang, F.F.M. (1973) A Statistical Summary of the Cause and Cost of Bridge Failures. FHWA RD-75-87. Federal Highway Administation, Washington, DC.

3. Shirole, A.M. Holt, RC (1991) Planning a Comprehensive Bridge Assurance Program. TRB 1290. Transportation Research Board, Washington, DC.

4. Peck, R., Hanson, W, Thornburn, T. (1974) Foundation Engineering, Second Edition, John Wiley & Sons Inc.

5. Fang, H. (1991) Foundation Engineering Handbook: Second Edition, Chapter 1. Van Nostrand Reinhold.

6. Hertlein, B. (1991) Dynamic Tests for Drilled Shafts. Short course notes. Texas A & M, August 1991.

7. Meunow, R. (1991) Personal Communication and notes from Maryland SHA, Scour Meeting, May 1991.

8. Gorin, S.R., Haeni, F.P. (1989) "Use of Surface Geophysical Methods to Assess Riverbed Scour at Bridge Piers." US Geological Survey, Water Resource Investigations Report 88-4212. Hartford, Conn.

9. Maryland State Highway Administration (1990) The Scour Priority Number. Bridge Scour Program. MDSHA, Baltimore, MD.

10. NCDOT (1990) Structure Scour Evaluation Plan for Existing Structures, NCDOT, Raleigh, N.C.

11. Richardson, E.V., Huber, F.W. (1991) Evaluation of Bridge Vulnerability to Hydraulic Forces, Stream Instability and Scour. TRB 1290. Transportation Research Board. Washington, D.C.

12. Moulton K, et al (1985) Tolerable Movement Criteria for Highway Bridges. FHWA RD-85-107. Federal Highway Administration, Washington, DC.

13. VDOT (1980) Drainage Manual.

14. AASHTO (1977) A Manual on User Benefit Analysis on Highway and Bus-Transit Improvements. Washington, D.C.

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