GEORGIA INSTITUTE OF TECHNOLOGY
OFFICE OF CONTRACT ADMINISTRATION
SPONSORED PROJECT INITIATION

Date: 12/18/80

Project Title: Development of Methodology for the Design and Construction of Stone Columns

Project No: E-20-686

Project Director: Dr. Barksdale

Sponsor: U.S. DOT Federal Highway Administration

Agreement Period: From 9/1/80 Until ~~3/1/82~~ 9/15/82

Type Agreement: Contract No. DTFH61-80-C-00111

Amount: $60,000 (E-20-686)
$ 9,516 (E-20-354)
$69,516 TOTAL

Reports Required: Monthly; Travel Plan; Interim; Load-Testing Program; Final

Sponsor Contact Person (s):

Technical Matters

Mr. Jerome DiMaggio
Federal Highway Administration
Office of Research
Materials Division
Washington, D.C. 20590

Contractual Matters
(thru OCA)

Mr. Luther D. McCollum
U.S. Dep't of Transportation
Federal Highway Administration
Office of Contracts and Procurement
Washington, D.C. 20590

Defense Priority Rating:

Assigned to: Civil Engineering (School/~~Laboratory~~)

COPIES TO:

- Project Director
- Division Chief (EES)
- School/Laboratory Director
- Dean/Director—EES
- Accounting Office
- Procurement Office
- Security Coordinator (OCA)
- ✓Reports Coordinator (OCA)
- Library, Technical Reports Section
- EES Information Office
- EES Reports & Procedures
- Project File (OCA)
- Project Code (GTRI)
- Other OCA Research Property Coordinator
Project Code (OCA)

GEORGIA INSTITUTE OF TECHNOLOGY **OFFICE OF CONTRACT ADMINISTRATION**

SPONSORED PROJECT TERMINATION SHEET

Date 4/15/83

Project Title: Development of Methodology for the Design and Construction of Stone Columns

Project No: E-20-686

Project Director: Dr. Barksdale

Sponsor: U.S. DOT Federal Highway Administration

Effective Termination Date: 8/31/82

Clearance of Accounting Charges: 8/31/82 (reports)

Grant/Contract Closeout Actions Remaining:

- [x] Final Invoice and Closing Documents
- [ ] Final Fiscal Report
- [ ] Final Report of Inventions
- [x] Govt. Property Inventory & Related Certificate
- [ ] Classified Material Certificate
- [ ] Other ______________________

Assigned to: Civil Engineering (School/~~Laboratory~~)

COPIES TO:

Administrative Coordinator
Research Property Management
Accounting
Procurement/EES Supply Services
Research Security Services
Reports Coordinator (OCA)
Legal Services (OCA)
Library
EES Public Relations (2)
Computer Input
Project File
Other Barksdale

LIBRARY DOES NOT HAVE

Monthly Progress Report No. 2, Oct 1 through Oct 31, 1980

Monthly Progress Report No. 3, Nov 1 through Nov 30, 1980

Monthly Progress Report No. 6, Feb 1 through Feb 28, 1981

Monthly Progress Report No. 14, Oct 1 through Oct 31, 1981

MONTHLY PROGRESS REPORT FEDERAL HIGHWAY ADMINISTRATION	Date of Report October 17, 1980

1 Project No.	2 Project Title	3
DTFH61-80-C-00111 GA TECH E20-686	Development of Methodology for the Design and Construction of Stone Columns	No. 1 From Sept. 1 To Sept. 30

4 Research Agency	5 Project Director(s)
Georgia Institute of Technology	Richard D. Barksdale, Director R. C. Bachus, Co-Director

6 Starting Date	7 Completion Date	8 % Time Expended	9 Schedule Status	10 Sufficiency of Funds
Sept.1, 1980	Phase I June 30, 1981	10%	☐ Ahead ☐ Behind ☒ On	☒ Sufficient ☐ Insufficient

Funds Authorized		Funds Expended				
11 Total (Phase I)	12 Current Fiscal Year	13 Total to Date	%	14 Current Fiscal Year	%	15 Report
FHWA – $49,924 GaTech – $9,516		$783 (travel)	2%	$783 (travel)	2%	None

16 Project Schedule

Research Tasks	Time Period (0 1 2 3 4 5 6 7 8 9 10)	% Task Completed
TASK A Current Practice		6%
TASK B Site Evaluation		0
TASK C Needed Research		0
Overall % Completed		

Approved Schedule Work Completed Schedule Projected Completion Schedule

17 Progress This Month (By Task)

Task A. 1. Literature Review was begun
2. U. S. organizations contacted for visit
3. Visits were made to Va. DOT, Vibroflotation Fdn. Co., FHWA, GKN Keller.

The project was just begun during the month with notice to proceed being received on approximately September 15, 1980.

Man Hours Expended: 60 man-hours

18 Work Planned for Next Month:

1. Continue literature review
2. Review theoretical approaches for predicting stone column behavior
3. Set up trip to Miss. DOT and Vibroflotation (Bob Mattox)
4. Discuss project with John Hughes.

19 Significant Technical Information, Recommendations, Implementation

None at this time

20 Problems

None at this time

21 Report Prepared by

Richard D. Barksdale | Dr. R. D. Barksdale | October 17, 1980

Signature | Name | Title

MONTHLY PROGRESS REPORT FEDERAL HIGHWAY ADMINISTRATION	Date of Report December 12, 1980

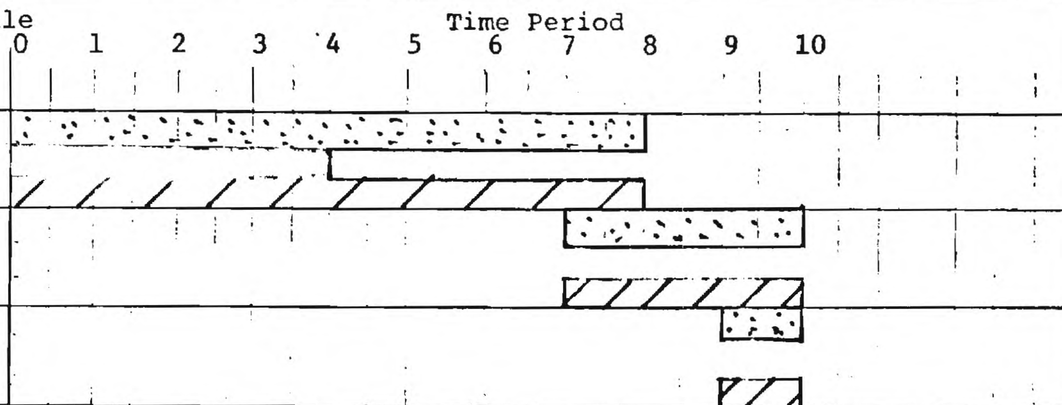
1 Project No.	2 Project Title	3
DTFH61-80-C-00111 GA TECH E20-686	Development of Methodology for the Design and Construction of Stone Columns	No. 4 From December 1 To December 31

4 Research Agency	5 Project Director(s)
Georgia Institute of Technology	Richard D. Barksdale, Director R. C. Bachus, Co-Director

6 Starting Date	7 Completion Date	8 % Time Expended	9 Schedule Status	10 Sufficiency of Funds
Sept. 1, 1980	Phase I June 30, 1981	40%	☐ Ahead ☐ Behind ☒ On	☒ Sufficient ☐ Insufficient

Funds Authorized		Funds Expended			
11 Total (Phase I)	-	13 Total to Date	%		15 Report
FHWA - $49,924 GaTech - $9,516		FHWA: $8961 GaTech: (0)	18%		Dec. 1980 - FCP Report, 43 p.

16 Project Schedule



Research Tasks	% Task Completed
TASK A Current Practice	50%
TASK B Site Evaluation	0
TASK C Needed Research	0
Overall % Completed	

Approved Schedule   Work Completed Schedule   Projected Completion Schedule

MONTHLY PROGRESS REPORT FEDERAL HIGHWAY ADMINISTRATION	Date of Report February 18, 1981

1 Project No.	2 Project Title	3
DTFH61-80-C-00111 GA TECH E20-686	Development of Methodology for the Design and Construction of Stone Columns	No. 5 From Jan. 1, 1981 To Jan. 31, 1981

4 Research Agency	5 Project Director(s)
Georgia Institute of Technology	Richard D. Barksdale, Director R. C. Bachus, Co-Director

6 Starting Date	7 Completion Date	8 % Time Expended	9 Schedule Status	10 Sufficiency of Funds
Sept. 1, 1980	Phase I June 30, 1981	50%	☐ Ahead ☐ Behind ☒ On	☒ Sufficient ☐ Insufficient

Funds Authorized		Funds Expended			
11 Total (Phase I)		13 Total to Date	%		15 Report
FHWA - $49,924 GaTech - $9,516		FHWA: $12,673 GaTech: (0)	25		Dec. 1980 - FCP Report, 43 p.

16 Project Schedule Research Tasks	Time Period 0 1 2 3 4 5 6 7 8 9 10	% Task Completed
TASK A Current Practice		60%
TASK B Site Evaluation		0
TASK C Needed Research		0
Overall % Completed		

Approved Schedule   Work Completed Schedule   Projected Completion Schedule

17 Progress This Quarter (By Task)

Task A. 1. Present talk at January Berkeley meeting.
2. Continue Santa Barbara finite element analysis; reduced material data for computer input; put data on code sheets to run.
3. Continue literature search; develop one or two page summaries of each paper read.
4. Finalize setting up trips to Europe and Asia.

Summary of Man Hours Expended: Task A

	Total	January
1. R.D. Barksdale, Project Director	220	45
2. R.C. Bachus, Co-Director	248	73
	468	118
3. Roger Blackwell, Graduate Student*	78	52

* No charge to project

Summary of Research Workshop on Ground Reinforcement attached.

18 Work Planned for Next Quarter

1. Continue literature review
2. Review theoretical approaches for predicting stone column behavior
3. Continue making computer runs for Santa Barbara
4. Begin Hampton, Va. finite element study.

19 Significant Technical Information, Recommendations, Implementation

None at this time

20 Problems

None at this time

21 Report Prepared by

Signature	Name	Title
	Dr. R. D. Barksdale	February 18, 1981

E20-686

MONTHLY PROGRESS REPORT FEDERAL HIGHWAY ADMINISTRATION	Date of Report April 13, 1981

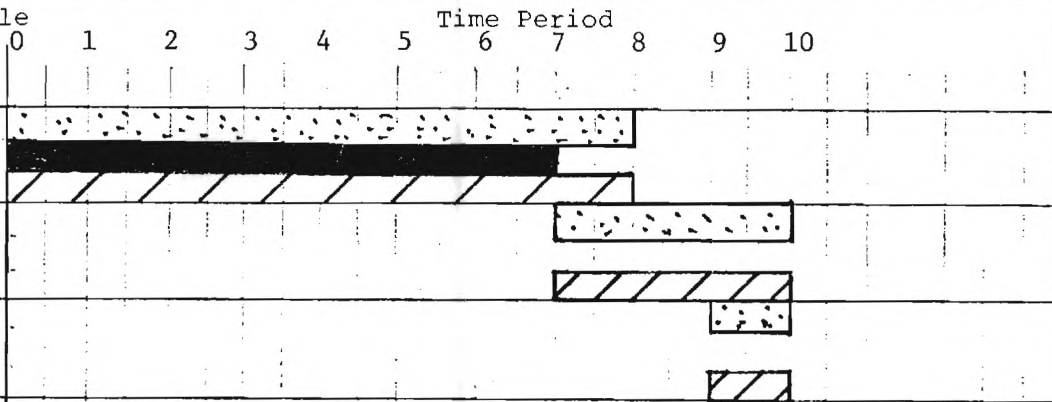
1 Project No.	2 Project Title	3
TFH61-80-C-00111 A TECH E20-686	Development of Methodology for the Design and Construction of Stone Columns	No. 7 From March 1, 1981 To March 31, 1981

4 Research Agency	5 Project Director(s)
Georgia Institute of Technology	Richard D. Barksdale, Director R. C. Bachus, Co-Director

6 Starting Date	7 Completion Date	8 % Time Expended	9 Schedule Status	10 Sufficiency of Funds
Sept.1, 1980	Phase I June 30, 1981	70% (Phase I)	☐ Ahead ☐ Behind ☒ On	☒ Sufficient ☐ Insufficient

Funds Authorized		Funds Expended			
1 Total (Phase I) FHWA - $49,924 aTech - $9,516		13 Total to Date FHWA: $28,328 GaTech: (0)	% 57		15 Report None

6 Project Schedule



Research Tasks	% Task Completed
ASK A urrent Practice	80%
ASK B ite Evaluation	0
ASK C eeded Research	0
verall % ompleted	

Approved Schedule   Work Completed Schedule   Projected Completion Schedule

17 Progress This Month (By Task)

Task A. 1. Continue Santa Barbara finite element analysis; reduced material data for computer input; put data on code sheets to run.
2. Continue literature search; develop one or two page summaries of each paper read.

Summary of Man Hours Expended: Task A

	Total	March
1. R.D. Barksdale, Project Director	450	180
2. R.C. Bachus, Co-Director	394	73
	844	233
3. Roger Blackwell, Graduate Student*	182	52
Total:	1026	285

* No charge to project

18 Work Planned for Next Month

1. Continue literature review
2. Review theoretical approaches for predicting stone column behavior
3. Continue making computer runs for Santa Barbara
4. Travel to Europe (R. C. Bachus)

19 Significant Technical Information, Recommendations, Implementation

Sand compaction piles are used extensively in Japan for slope stability of fills and embankments, usually for reclaimed land projects. Construction costs (in the U.S.) would appear to be about 1/2 of that for stone columns. A good potential appears to exist for using sand compaction piles for highway embankments where stability is of major concern. In Japan 80% of the settlement is assumed to have occurred by the time embankment construction is complete.

20 Problems

None at this time

21 Report Prepared by:

Signature — Dr. R. D. Barksdale (Name) — April 13, 1981 (Title)

E20-686

MONTHLY PROGRESS REPORT

FEDERAL HIGHWAY ADMINISTRATION

Date of Report: June 5, 1981

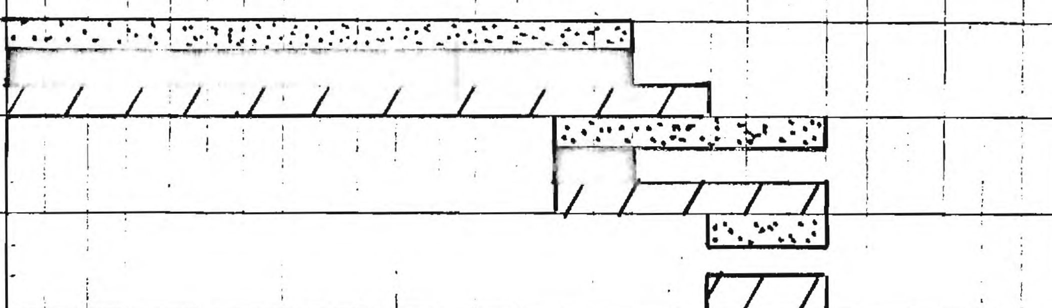
1 Project No. DTFH61-80-C-00111 GA TECH E20-686	2 Project Title Development of Methodology for the Design and Construction of Stone Columns	3 Report No. 8 From April 1, 1981 To April 30, 1981

4 Research Agency Georgia Institute of Technology	5 Project Director(s) Richard D. Barksdale, Director Robert C. Bachus, Co-Director

6 Starting Date	7 Completion Date	8 % Time Expended	9 Schedule Status	10 Sufficiency of Funds
Sept. 1, 1980	Phase I June 30, 1981	80%	☐ Ahead ☐ Behind ☒ On	☒ Sufficient ☐ Insufficient

Funds Authorized		Funds Expended			
11 Total (Phase I) FHWA $49,924 GaTech $9,516		13 Total to Date FHWA: $27,580 GaTech: 0	% 55 0		15 Report Quarter $3,027.00

16 Project Schedule — Time Period 0 1 2 3 4 5 6 7 8 9 10



Research Tasks	% Task Completed
TASK A Current Practice	80
TASK B Site Evaluation	30
TASK C Needed Research	0
Overall % Completed	

Approved Schedule · Work Completed Schedule · Projected Completion Schedule

17 Progress This Month (By Task)

Task A. 1. Continue Finite Element Analysis: Santa Barbara
2. Develop Summary of Asia and Europe Trip
3. Develop Model Study Plan

Summary of Man-Hours Expended: Task A

	Total	April
1. R.D. Barksdale, Project Director	495	45
2. R. C. Bachus, Co-Director	524	130
3. Roger Blackwell*, Graduate Student	234	52
Total:	1253	227

* No charge to project.

18 Work Planned for Next Month

1. Continue work on trip summaries.
2. Begin work on Task B and C.

19 Significant Technical Information, Recommendations, Implementation

None at this time

20 Problems

None at this time

Report Pr...

Signature | Dr. R. D. Barksdale (Name) | June 5, 1981 (Title)

MONTHLY PROGRESS REPORT FEDERAL HIGHWAY ADMINISTRATION				Date of Report June 22, 1981
1 Project No. DTFH61-80-C-00111 GA TECH E20-686	2 Project Title Development of Methodology for the Design and Construction of Stone Columns			3 Report No. 9 From May 1, 1981 To May 31, 1981
4 Research Agency Georgia Institute of Technology			5 Project Director(s) Richard D. Barksdale, Director Robert C. Bachus, Co-Director	
6 Starting Date Sept. 1, 1980	7 Completion Date Phase I June 30, 1981	8 % Time Expended 90%	9 Schedule Status ☐ Ahead ☐ Behind ☒ On	10 Sufficiency of Funds ☒ Sufficient ☐ Insufficient

Funds Authorized		Funds Expended			
11 Total (Phase I) FHWA $49,924 GaTech $9,516		13 Total to Date FHWA: $32,006 GaTech: 0	% 64 0		15 Report Month $4,426.00

16 Project Schedule

Research Tasks	Time Period 0 1 2 3 4 5 6 7 8 9 10	% Task Completed
TASK A Current Practice		90
TASK B Site Evaluation		50
TASK C Needed Research		0
Overall % Completed		

Approved Schedule   Work Completed Schedule   Projected Completion Schedule

17 Progress This Month (By Task)

Task A. 1. Develop Summary of Asia and Europe Trip
2. Develop Model Study Plan

Task B. 1. Begin Site Evaluation

Summary of Man-Hours Expended: Task A

	Total	May
1. R.D. Barksdale, Project Director	540	45
2. R. C. Bachus, Co-Director	564	40
3. Roger Blackwell*, Graduate Student	286	52
Total:	1390	137

* No charge to project.

18 Work Planned for Next Month

1. Continue work on trip summaries.
2. Work on Task B and C.

19 Significant Technical Information, Recommendations, Implementation

None at this time

20 Problems

None at this time

Report Prepared by:

Signature — Dr. R. D. Barksdale (Name) — June 24, 1981 (Title)

MONTHLY PROGRESS REPORT FEDERAL HIGHWAY ADMINISTRATION	Date of Report September 8, 1981

1 Project No.	2 Project Title	3 Report No. 10
TFH61-80-C-00111 A TECH· E20-686	Development of Methodology for the Design and Construction of Stone Columns	From June 1, 1981 To June 30, 1981

4 Research Agency	5 Project Director(s)
Georgia Institute of Technology	Richard D. Barksdale, Director Robert C. Bachus, Co-Director

6 Starting Date	7 Completion Date	8 % Time Expended	9 Schedule Status	10 Sufficiency of Funds
Sept. 1, 1980	Phase II February 30, 1982	55%	☐ Ahead ☐ Behind ☒ On	☒ Sufficient ☐ Insufficient

Funds Authorized		Funds Expended			
1 Total		13 Total to Date	%		15 Report Month
FHWA $97,800 GaTech $18,300		FHWA: $32,529 GaTech: 0	33 0		$3,274

6 Project Schedule Research Tasks	Time Period 0 1 2 3 4 5 6 7 8 9 10	% Task Completed
ASK A rrent Practice		90
ASK B te Evaluation		90
ASK C eded Research		90
erall % mpleted		90

Approved Schedule   Work Completed Schedule   Projected Completion Schedule

17 Progress This Month (By Task)

Task A. 1. Write summary of Asia and Europe Trips
2. Develop Model Study Plan

Task B. 1. Begin Site Evaluation

Summary of Man-Hours Expended: Task A

	Total	June
1. R.D. Barksdale, Project Director	690	150
2. R. C. Bachus, Co-Director	626	62
3. Roger Blackwell , Graduate Student	342	56
Total:	1658	268

18 Work Planned for Next Month

1. Begin model test design
2. Continue writing Phase I draft report
3. Work on Finite Element Design Curves

19 Significant Technical Information, Recommendations, Implementation

None at this time

20 Problems

None at this time

21 Report Prepared by:

Signature	Dr. R. D. Barksdale Name	Project Director/Professor Title

MONTHLY PROGRESS REPORT FEDERAL HIGHWAY ADMINISTRATION	Date of Report September 8, 1981

1 Project No.	2 Project Title	3 Report No. 11
TFH61-80-C-00111 A TECH· E20-686	Development of Methodology for the Design and Construction of Stone Columns	From July 1, 1981 To July 31, 1981

4 Research Agency	5 Project Director(s)
Georgia Institute of Technology	Richard D. Barksdale, Director Robert C. Bachus, Co-Director

6 Starting Date	7 Completion Date	8 % Time Expended	9 Schedule Status	10 Sufficiency of Funds
Sept. 1, 1980	Phase I & II February 30, 1982	61%	☐ Ahead ☐ Behind ☒ On	☒ Sufficient ☐ Insufficient

Funds Authorized		Funds Expended			
1 Total (Phase I) FHWA $97,800 GaTech $18,300		13 Total to Date FHWA: $38,287 GaTech: 0	% 39 0		15 Report Month $5,758

6 Project Schedule — Phase I — Time Period 0 1 2 3 4 5 6 7 8 9 10

Research Tasks	% Task Completed
ASK A urrent Practice	100
ASK B ite Evaluation	100
ASK C eeded Research	100
erall % mpleted	100

Approved Schedule   Work Completed Schedule   Projected Completion Schedule

17 Progress This Month (By Task)

Task A. 1. Complete report on Asia and Europe Trip
2. Design Model
Task B. 1. Complete Site Evaluation

Summary of Man-Hours Expended:

	Total	July
1. R.D. Barksdale, Project Director	830	140
2. R. C. Bachus, Co-Director	676	50
3. Roger Blackwell , Graduate Student	440	98
Total:	1946	288

18 Work Planned for Next Month

1. Continue finite element design curve study
2. Begin building model

19 Significant Technical Information, Recommendations, Implementation

None at this time

20 Problems

None at this time

21 Report Prepared by

Dr. R. D. Barksdale — September 8, 1981

Signature — Name — Title

MONTHLY PROGRESS REPORT

FEDERAL HIGHWAY ADMINISTRATION

Date of Report

Sept. 8, 1981

Project No.	2 Project Title	3 Report No. 12
H61-80-C-00111 ECH· E20-686	Development of Methodology for the Design and Construction of Stone Columns	From Aug. 1, 1981 To Aug. 31, 1981

Research Agency	5 Project Director(s)
Georgia Institute of Technology	Richard D. Barksdale, Director Robert C. Bachus, Co-Director

Starting Date	7 Completion Date	8 % Time Expended	9 Schedule Status	10 Sufficiency of Funds
ept. 1, 1980	Phase I & II Feb. 31, 1982	67%	☐ Ahead ☐ Behind ☒ On	☒ Sufficient ☐ Insufficient

unds Authorized		Funds Expended			
Total (Phase & II) HWA $97,800 aTech $18,300		13 Total to Date FHWA: $51,000 GaTech: 0	% 52 0		15 Report Month $7,255

Project Schedule — Phase II — Time Period

Research Tasks	0 1 2 3 4 5 6 7 8	% Task Completed
A Study		5
B gn Methodology		25
ll % eted		

Approved Schedule   Work Completed Schedule   Projected Completion Schedule

17 Progress This Month (By Task)

Task A. 1. Complete Design of Models
2. Fabricate Model Tank

Task B. 1. Continue Finite Element Development of Design Curves

Summary of Man-Hours Expended: Task A

	Total	August
1. R.D. Barksdale, Project Director	980	150
2. R. C. Bachus, Co-Director	741	65
3. Roger Blackwell , Graduate Student	544	104
Total:	2265	319

8 Work Planned for Next Month

1. Complete Model Tank
2. Begin Model Tests
3. Continue Finite Element Work

9 Significant Technical Information, Recommendations, Implementation

See Attached Preliminary Design Curves

0 Problems

None at this time

Report Prepared by

	Dr. R. D. Barksdale	September 8, 1981
Signature	Name	Title

PRELIMINARY DESIGN CURVES

FOR

WORKING LOAD BASED ON LINEAR ELASTIC THEORY

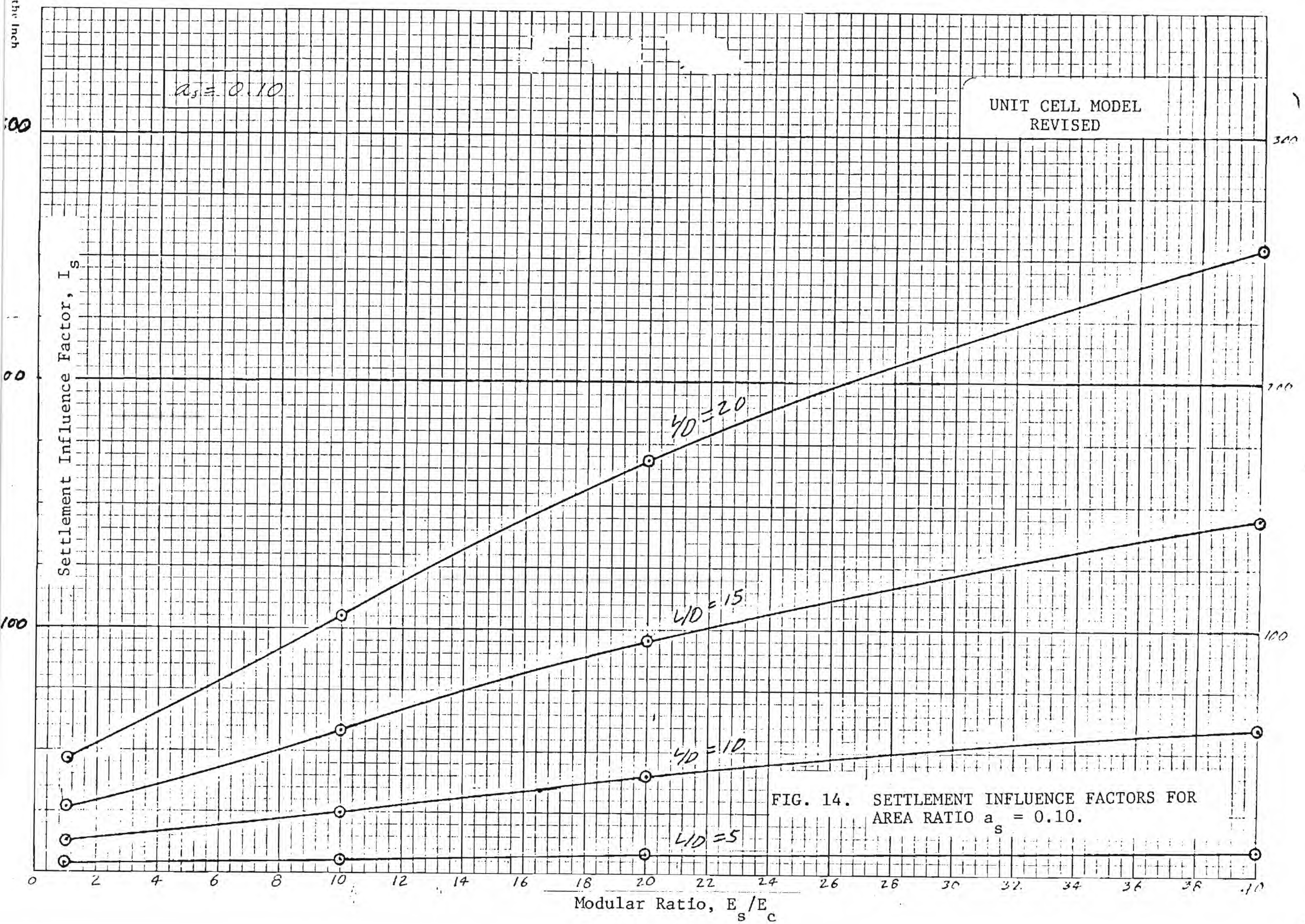


FIG. 14. SETTLEMENT INFLUENCE FACTORS FOR AREA RATIO $a_s = 0.10$.

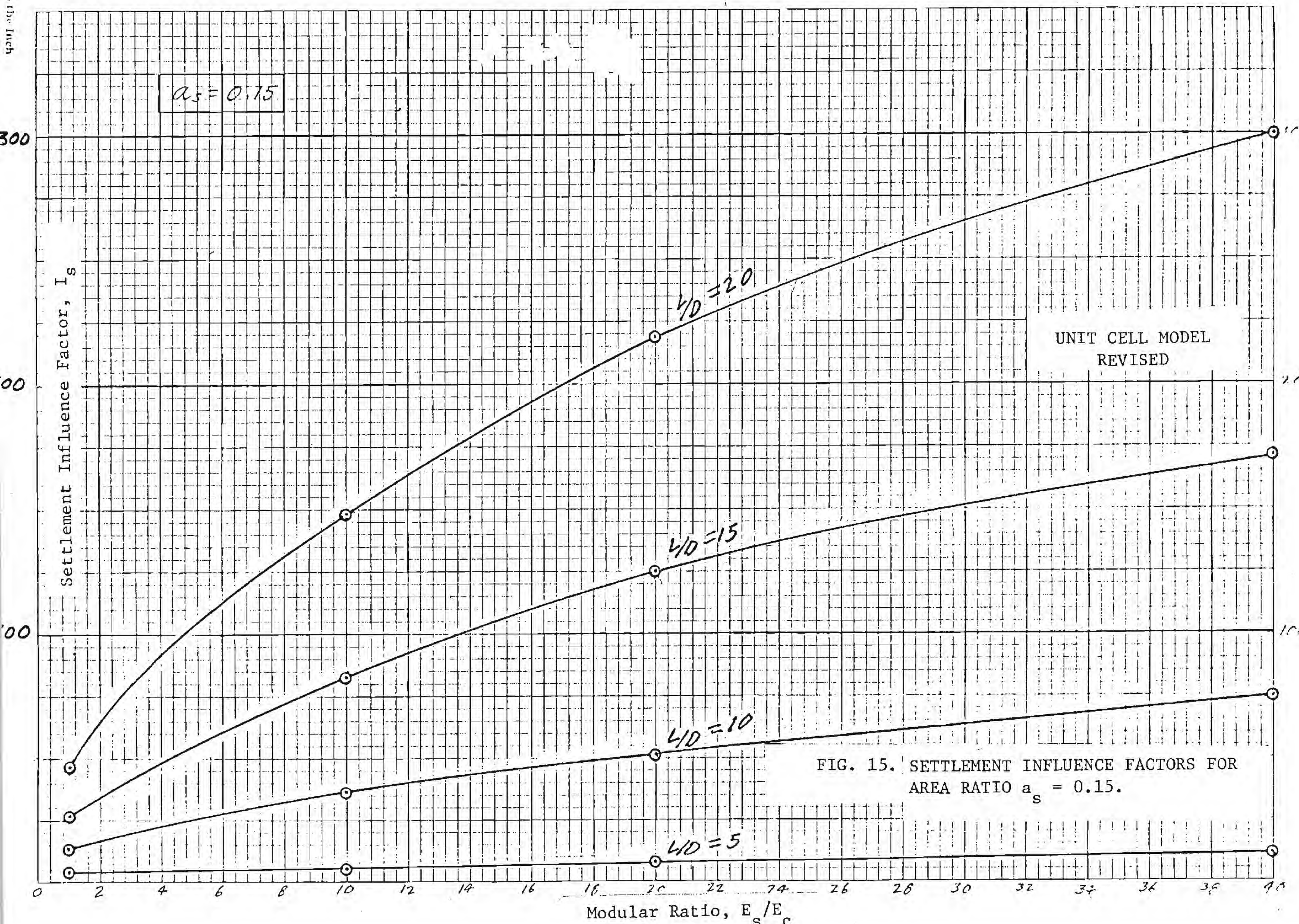


FIG. 15. SETTLEMENT INFLUENCE FACTORS FOR AREA RATIO a_s = 0.15.

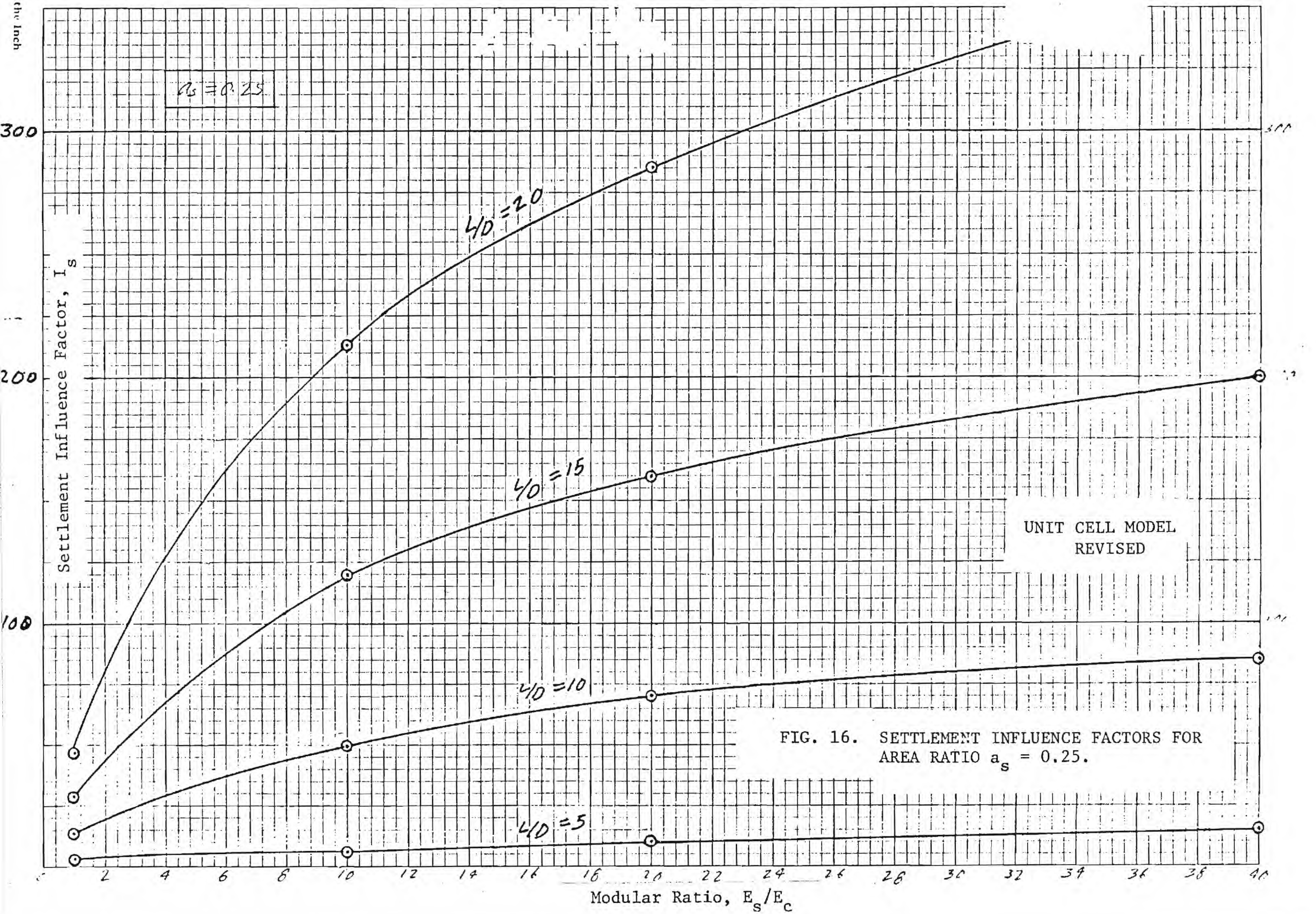
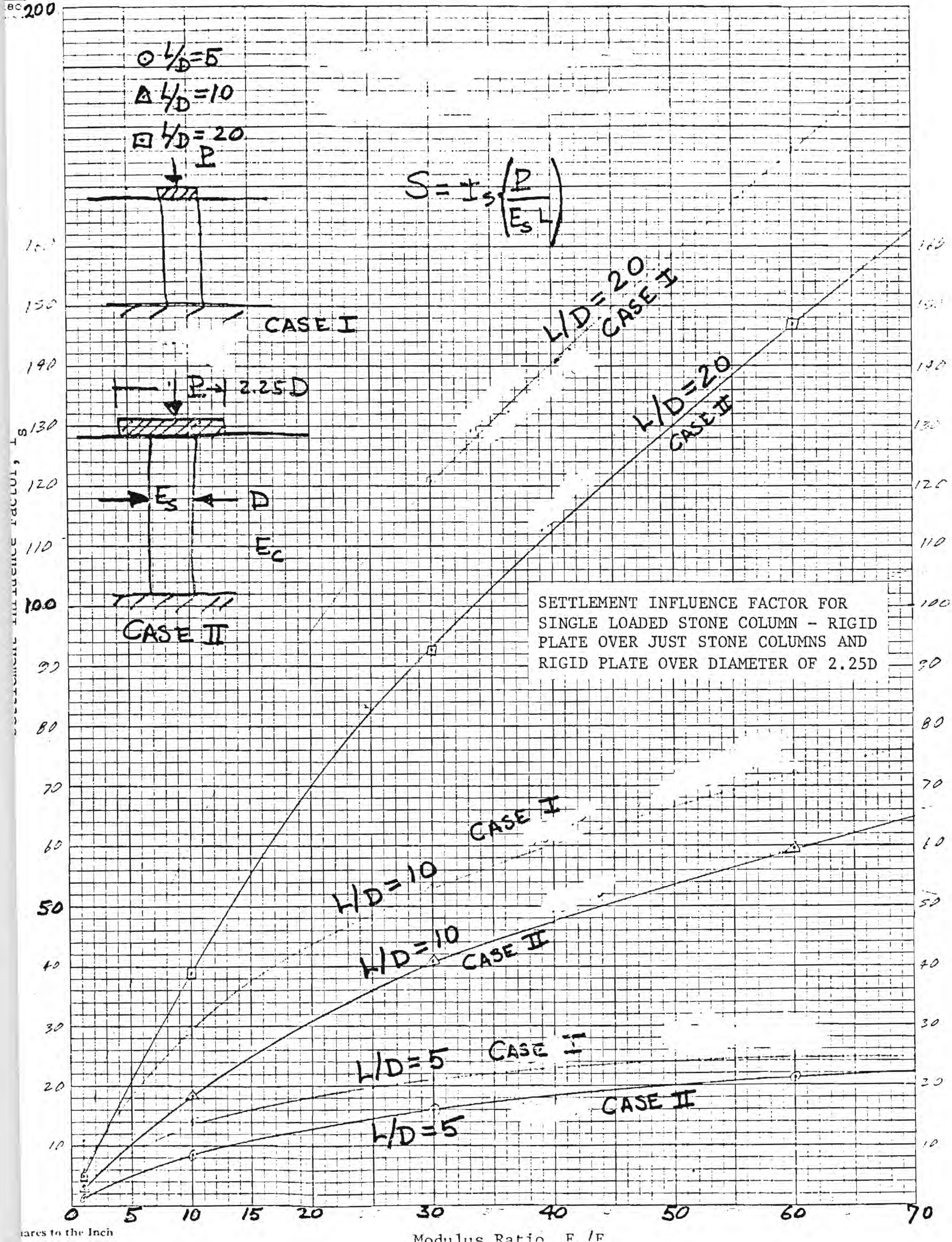
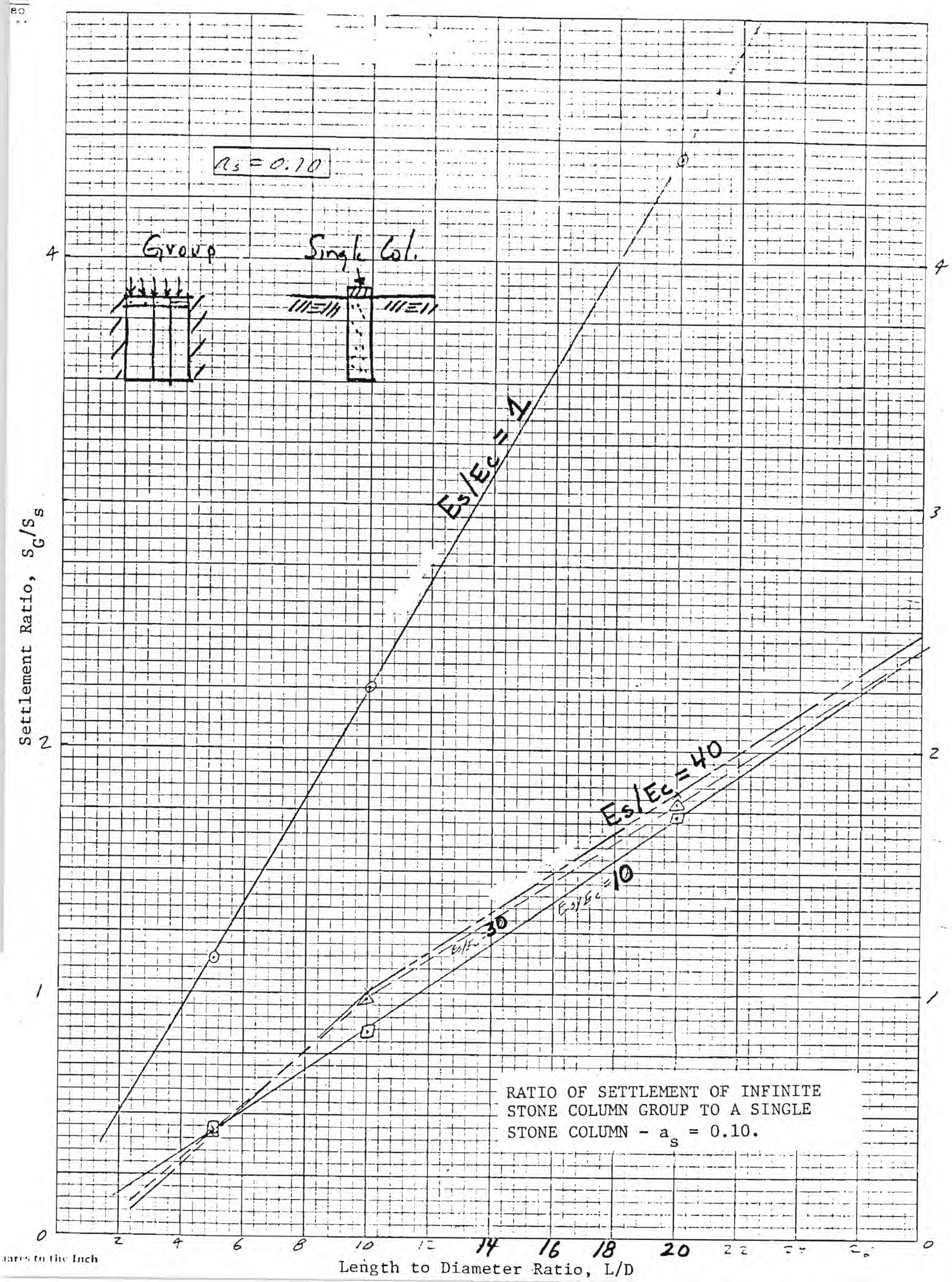


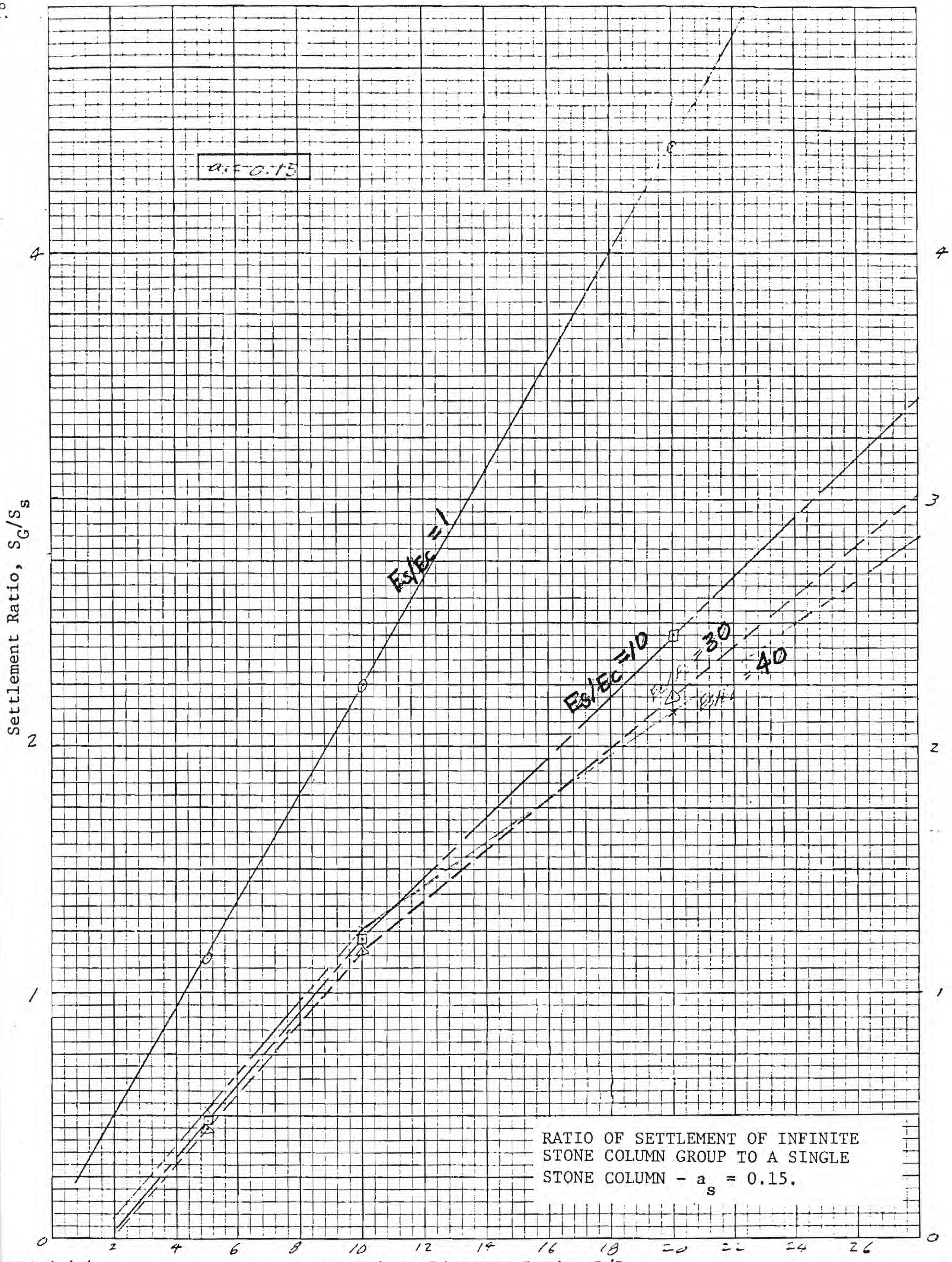
FIG. 16. SETTLEMENT INFLUENCE FACTORS FOR AREA RATIO $a_s = 0.25$.



⊙ L/D=5
△ L/D=10
⊡ L/D=20
P
S = I_s (P / E_s L)
CASE I
P
2.25D
E_s
D
E_c
CASE II
L/D=20 CASE I
L/D=20 CASE II
L/D=10 CASE I
L/D=10 CASE II
L/D=5 CASE I
L/D=5 CASE II
SETTLEMENT INFLUENCE FACTOR FOR SINGLE LOADED STONE COLUMN - RIGID PLATE OVER JUST STONE COLUMNS AND RIGID PLATE OVER DIAMETER OF 2.25D
Settlement Influence Factor, I_s
Modulus Ratio, E_c/E_s

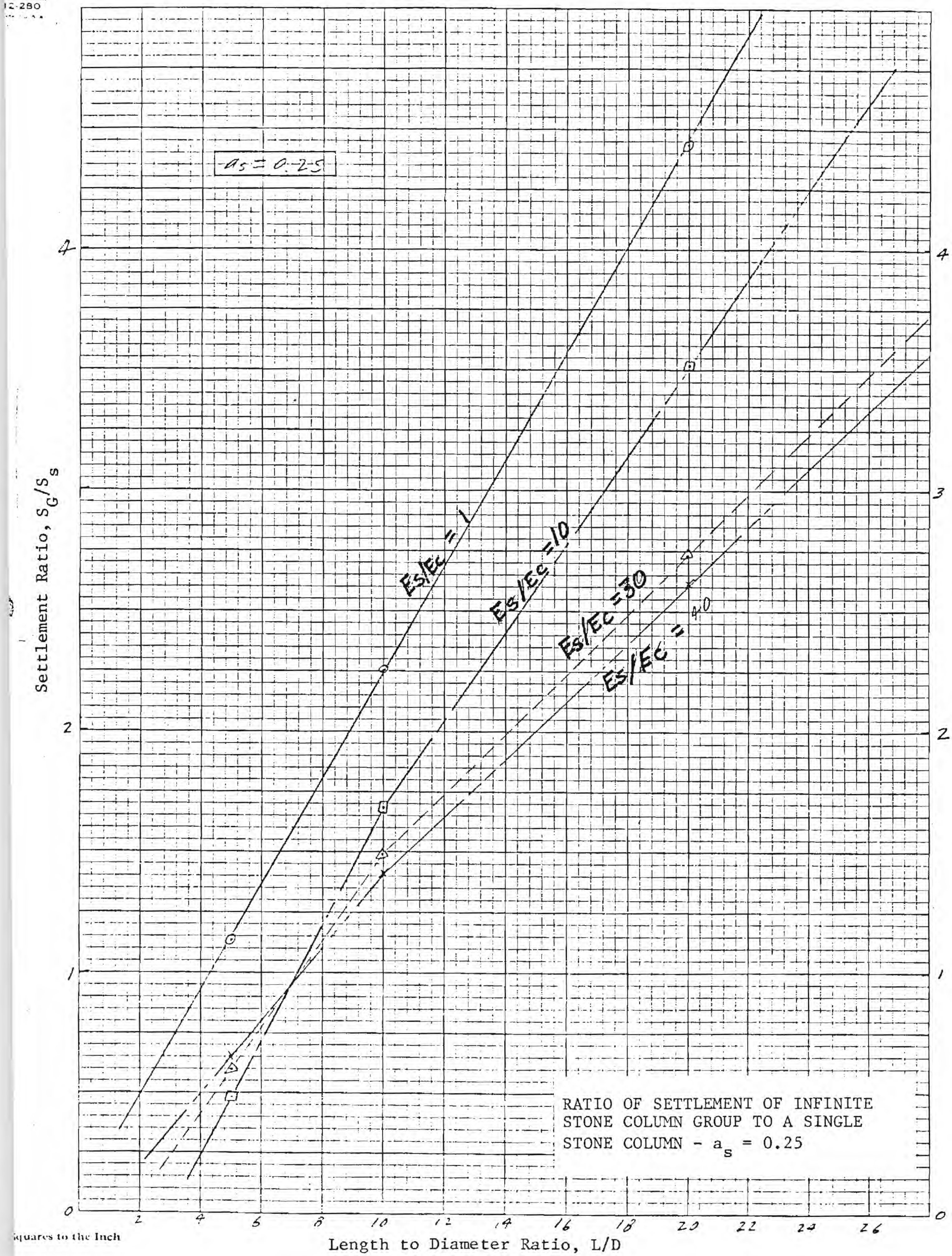


$a_s = 0.10$
Group
Single Col.
Es/Ec = 1
Es/Ec = 40
Es/Ec = 30
Es/Ec = 10
RATIO OF SETTLEMENT OF INFINITE STONE COLUMN GROUP TO A SINGLE STONE COLUMN - a_s = 0.10.
Settlement Ratio, S_G/S_s
Length to Diameter Ratio, L/D



RATIO OF SETTLEMENT OF INFINITE STONE COLUMN GROUP TO A SINGLE STONE COLUMN - a_s = 0.15.

Length to Diameter Ratio, L/D



Squares to the Inch

MONTHLY PROGRESS REPORT FEDERAL HIGHWAY ADMINISTRATION	Date of Report Sept. 8, 1981

1 Project No.	2 Project Title	3 Report No. 13
DTFH61-80-C-00111 GA TECH· E20-686	Development of Methodology for the Design and Construction of Stone Columns	From Sept. 1, 1981 To Sept. 31, 1981

4 Research Agency	5 Project Director(s)
Georgia Institute of Technology	Richard D. Barksdale, Director Robert C. Bachus, Co-Director

6 Starting Date	7 Completion Date	8 % Time Expended	9 Schedule Status	10 Sufficiency of Funds
Sept. 1, 1980	Phase I & II Feb. 31, 1982	72%	☐ Ahead ☒ Behind ☐ On	☒ Sufficient ☐ Insufficient

Funds Authorized	Funds Expended			
11 Total (Phase & II)	13 Total to Date	%		15 Report Month
FHWA $97,800 GaTech $18,300	FHWA: $55,619 GaTech: 0	57 0		$4,619

16 Project Schedule Research Tasks	Phase II Time Period 0 1 2 3 4 5 6 7 8	% Task Completed
TASK A Lab Study		20
TASK B Design Methodology		38
Overall % Completed		25

Approved Schedule   Work Completed Schedule   Projected Completion Schedule

17 Progress This Month (By Task)

Task A. 1. Complete model tank fabrication
2. Begin model tests/trial

Task B. 1. Continue Finite Element Development of Design Curves and Study Vibroflotation Analysis; Investigate field direct shear test.

Summary of Man-Hours Expended: Task A

	Total	Sept.
1. R.D. Barksdale, Project Director	1035	55
2. R. C. Bachus, Co-Director	799	58
3. Roger Blackwell, Graduate Student	606	62
4. Brent Reid, Student	47	47
Total:	2487	222

18 Work Planned for Next Month

1. Continue model tests
2. Continue Finite Element Work
3. Continue evaluation of field direct shear test
4. Continue evaluation of Vibroflotation Co. analysis method

19 Significant Technical Information, Recommendations, Implementation

None at this time

20 Problems

None at this time

21 Report Prepared by:

Signature	Dr. R. D. Barksdale	Project Director
Signature	Name	Title

E20-686

MONTHLY PROGRESS REPORT FEDERAL HIGHWAY ADMINISTRATION	Date of Report December 17, 1981

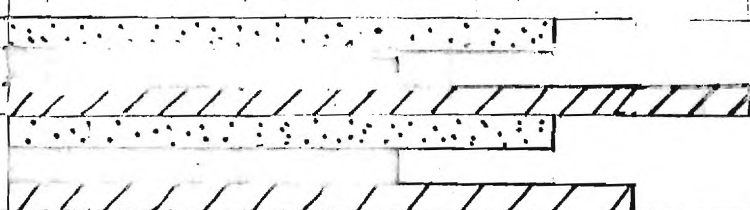
1 Project No.	2 Project Title	3 Report No. 15
DTFH61-80-C-00111 GA TECH E20-686	Development of Methodology for the Design and Construction of Stone Columns	From Nov. 1, 1981 To Nov. 30, 1981

4 Research Agency	5 Project Director(s)
Georgia Institute of Technology	Richard D. Barksdale, Director Robert C. Bachus, Co-Director

6 Starting Date	7 Completion Date	8 % Time Expended	9 Schedule Status	10 Sufficiency of Funds
Sept. 1, 1980	Phase I & II Feb. 31, 1982	79%	☐ Ahead ☒ Behind ☐ On	☒ Sufficient ☐ Insufficient

Funds Authorized		Funds Expended			
11 Total (Phase I & II)		13 Total to Date	%		15 Report Month
FHWA $97,800 GaTech $18,300		FHWA: $63,922 GaTech: $6468	65 35		$5,459 (Ga.Tech: $3234)

16 Project Schedule — Phase II — Time Period 0 1 2 3 4 5 6 7 8



Research Tasks	% Task Completed
TASK A Lab Study	30
TASK B Design Methodology	55
Overall % Completed	40

Approved Schedule — Work Completed Schedule — Projected Completion Schedule

17 Progress This Month (By Task)

Task A. 1. Fabricated Circular Model Tank
2. Continue Model Test 1

Task B. 1. Continue Finite Element Development of Design Curves and Study Vibroflotation Analysis; Investigate field direct shear test.

Summany of Man Hours Expended:

	Total	Nov.
R. D. Barksdale, Project Director	1281	123
R. C. Bachus, Co-Director	870	36
Roger Blackwell, Graduate Student	744	69
Brent Reid, Student	87	-
George Kaffezakis, Graduate Student	120	60
Steve Long, Machinist	70	-
Total:	3172	288

18 Work Planned for Next Month

1. Continue Rectangular Tank Model Test; Begin circular tank model test.
2. Continue Finite Element Work; Preliminary analysis of Hampton, Va. load tests.
3. Continue Evaluation of Field Direct Shear Test; Run model study in lab.
4. Continue Evaluation of Vibroflotation Co. Analysis Method.

19 Significant Technical Information, Recommendations, Implementation

Problems of under-estimation of settlement appear to exist when elastic finite element method is applied to very soft clays such as Hampton, Va.; the Japanese method appears to perhaps overpredict the settlement. (tentative)

20 Problems

Rubber membrane on rectangular model tank pulled out ruining the test. Model testing using slurry consolidated clay is just slow - but certainly the best approach. The bugs are about worked out of the test procedure.

Report Prepared by

Signature	Name	Title
	Dr. R. D. Barksdale	Project Director

MONTHLY PROGRESS REPORT

FEDERAL HIGHWAY ADMINISTRATION

Date of Report: Jan. 22, 1982

1 Project No.	2 Project Title	3 Report
DTFH61-80-C-00111 GA TECH · E20-686	Development of Methodology for the Design and Construction of Stone Columns	No. 16 From Dec. 1, 1981 To Dec. 31, 1981

4 Research Agency	5 Project Director(s)
Georgia Institute of Technology	Richard D. Barksdale, Director Robert C. Bachus, Co-Director

6 Starting Date	7 Completion Date	8 % Time Expended	9 Schedule Status	10 Sufficiency of Funds
Sept. 1, 1980	Phase I & II Feb. 31, 1982	88%	☐ Ahead ☒ Behind ☐ On	☒ Sufficient ☐ Insufficient

Funds Authorized	Funds Expended		
11 Total (Phase I & II)	13 Total to Date	%	15 Report Month
FHWA $97,800 GaTech $18,300	FHWA:$69,178 GaTech:$9,702	71 53	$5,256 (Ga.Tech: $3234)

16 Project Schedule — Phase II — Time Period

Research Tasks	0 1 2 3 4 5 6 7 8	% Task Completed
TASK A Lab Study		35
TASK B Design Methodology		65
Overall % Completed		50

Approved Schedule   Work Completed Schedule   Projected Completion Schedule

17 Progress This Month (By Task)

Task A. On the rectangular model test box the rubber membrane which caused a model test failure the month before was replaced with a heavier member having holes with close tolerances. A new sample was consolidated from a slurry and the test is presently being repeated. The cylindrical model has been completed and a sample settled out in the cylinder. An improved technique was developed for placing the lead shot in the model tanks. The loading system for the cylinder has been designed and is being fabricated.

Task B. 1. Finished elastic settlement curves for stone columns. Settlement curves underpredict settlement as presently used in very soft clays due to yield of the stone because of large σ_1/σ_3 ratio. Nonlinear finite element analyses are being initiated to investigate this further and develop new curves or modify as required existing curves.

2. Field load test evidence exists indicating a bearing failure can develop in the soft clay behind a stone column during a direct shear test if the test is not performed to prevent it. Theoretical equations have been developed to predict the effectively reduced shear strength of stone column due to this type failure. These results indicate that the "effective" ϕ of the stone column varies with overburden pressure, clay shear strength and width of the column in addition to strength of the stone column. Probably for a very soft clay a limiting ϕ exists in the field for usual conditions.

18 Work Planned for Next Month

1. Begin nonlinear finite element runs for modifying elastic charts or developing new ones.
2. Review and check theoretical equations for bearing failure behind a stone column.
3. Continue laboratory evaluation of Hampton, Va. clay and clayey sand properties.
4. Continue model tests.

19 Significant Technical Information, Recommendations, Implementation

1. Bearing failure of stone column can occur when laterally loaded.
2. Important yielding of stone occurs in soft clays result (refer to Item 17)

20 Problems

1. Continuing problem of very slow consolidation of slurry and slow loading of column for drained test.
2. Elastic settlement curves must be modified for very soft clays.

21 Report Prepared by

	Dr. R. D. Barksdale	Project Director
Signature	Name	Title

E-20-686

MONTHLY PROGRESS REPORT

FEDERAL HIGHWAY ADMINISTRATION

Date of Report: March 8, 1982

1 Project No.	2 Project Title	3 Report No. 17
DTFH61-80-C-00111 GA TECH· E20-686	Development of Methodology for the Design and Construction of Stone Columns	From Jan. 1, 1981 To Jan. 31, 1981

4 Research Agency	5 Project Director(s)
Georgia Institute of Technology	Richard D. Barksdale, Director Robert C. Bachus, Co-Director

6 Starting Date	7 Completion Date	8 % Time Expended	9 Schedule Status	10 Sufficiency of Funds
Sept. 1, 1980	Phase I & II Feb. 31, 1982	94%	☐ Ahead ☒ Behind ☐ On	☒ Sufficient ☐ Insufficient

Funds Authorized	Funds Expended		
11 Total (Phase I & II)	13 Total to Date	%	15 Report Month
FHWA $97,800 GaTech $18,300	FHWA: $72,137 GaTech: $9,702	74 (53)	$2,959 (Ga.Tech: $0)

16 Project Schedule

Research Tasks	Phase II — Time Period 0 1 2 3 4 5 6 7 8	% Task Completed
TASK A Lab Study		40
TASK B Design Methodology		70
Overall % Completed		55

Approved Schedule — Work Completed Schedule — Projected Completion Schedule

17 Progress This Month (By Task) (Refer to ATTACHMENT NO. 1 for Item 17)

(For Time Expended Refer to Table 1)

18 Work Planned for Next Month

Task A. Run a second x-ray test. Begin mass production of design data using 24 in. dia. circular tanks.

Task B. Run elastic (and nonlinear if desirable) analysis of unit cell model to establish the effect of lateral spreading on settlement.

19 Significant Technical Information, Recommendations, Implementation

Refer to 17 above.

20 Problems

Refer to 17 above.

21 Report Prepared by

Signature	Dr. R. D. Barksdale Name	Project Director Title

ATTACHMENT I

17. <u>Progress This Money By Task</u> (For Time Expended Refer to Table 1)

Task A. Load tests to failure (undrained) on a single column and three columns in a row were completed. The test was conducted by placing a rigid footing 2D by 2D in size on top of the single column and 2D by 5D in size on the three column group (D is the dia. of the column). Radiographs (Fig. 1 and Fig. 2) show the displacements of the stone columns. Presence of the footing over a wider area than just the column caused movements to be primarily downward. In both tests the X-ray did not show up in the very upper part of the column; hopefully this problem has been isolated and corrected.

Two circular tanks were fabricated and prepared for model tests. Clay will be placed in these tanks by hand and rammed in with a compactor at high water content. This approach will greatly increase production for developing design type charts for column groups. A rigid plate will be used in all tests.

Task B. Nonlinear finite element analyses were carried out for several typical material properties. Settlements calculated were about 30% greater than for the elastic case. Field settlements, however, are considered at this time to be in excess of the computer values.

The problem appears to be due to use of the unit cell model whereas in the field lateral spreading appears to have an important effect on reducing effective lateral confinement. The Jordan Road data certainly indicates this to be true (refer to Fig. 4a of Phase I, Draft Report, Site Improvement Using Stone Columns previously submitted to FHWA). A finite element analysis with a soft boundary that will allow lateral deflection is presently being conducted.

The explanation of the problem with the unit cell appears to be as follows:

1. The unit cell has rigid boundaries.
2. As load is taken by the stone column, the stone fails.
3. Upon failure the stone pushes laterally against the clay.
4. Vertical settlement in the clay can be expressed as

$$S_z^{clay} = \int \varepsilon_z dz$$

where ε_z is the vertical strain in the clay.

5. The vertical strain ε_z in the clay is expressed by elasticity as

$$\varepsilon_z = \frac{1}{E}[\sigma_z - \nu(\sigma_r + \sigma_\theta)]$$

6. Because of the large lateral stress in the clay (caused by the failed stone column and rigid boundary) the vertical

(cont).

6. strain in the clay is reduced to a value considerably less than for the constrained state as measured in a consolidation test.

7. The finite element analysis being performed with the soft boundary should help to clear up this important question.

Table 1

Summary of Man-Hours Expended

	Total	January	% By Task Task (Jan. Work) A	B
R. D. Barksdale, Project Director	1446	45	40%	60%
R. C. Bachus, Co-Director	935	35	100%	0
Roger Blackwell, Graduate Student	792	23	0	100%
Brent Reid, Student	112	10	100%	0
George Kaffezakis, Graduate Student	250	60	100%	0
Steve Long, Machinist	70	-	100%	0
Total	3605	173		

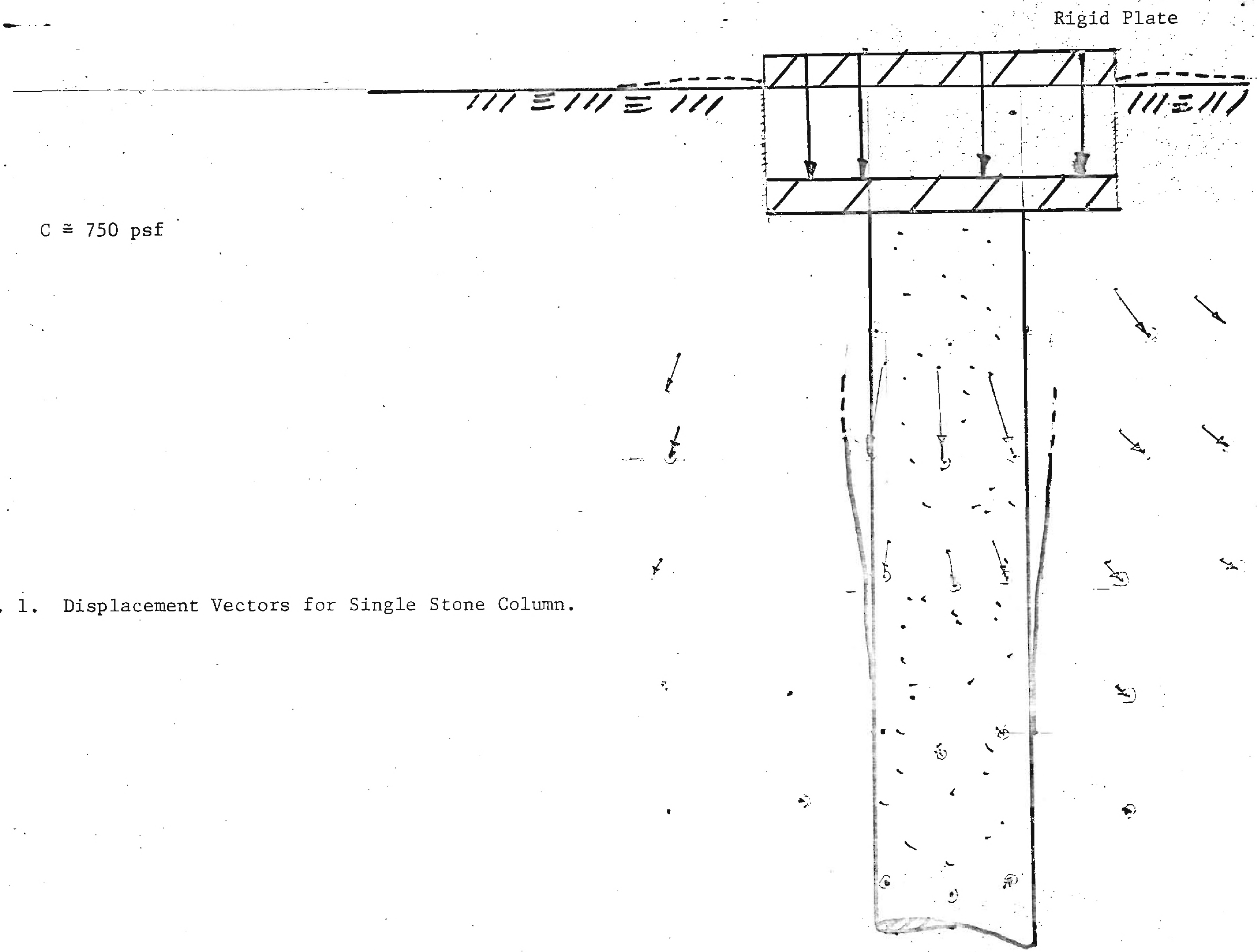


Fig. 1. Displacement Vectors for Single Stone Column.

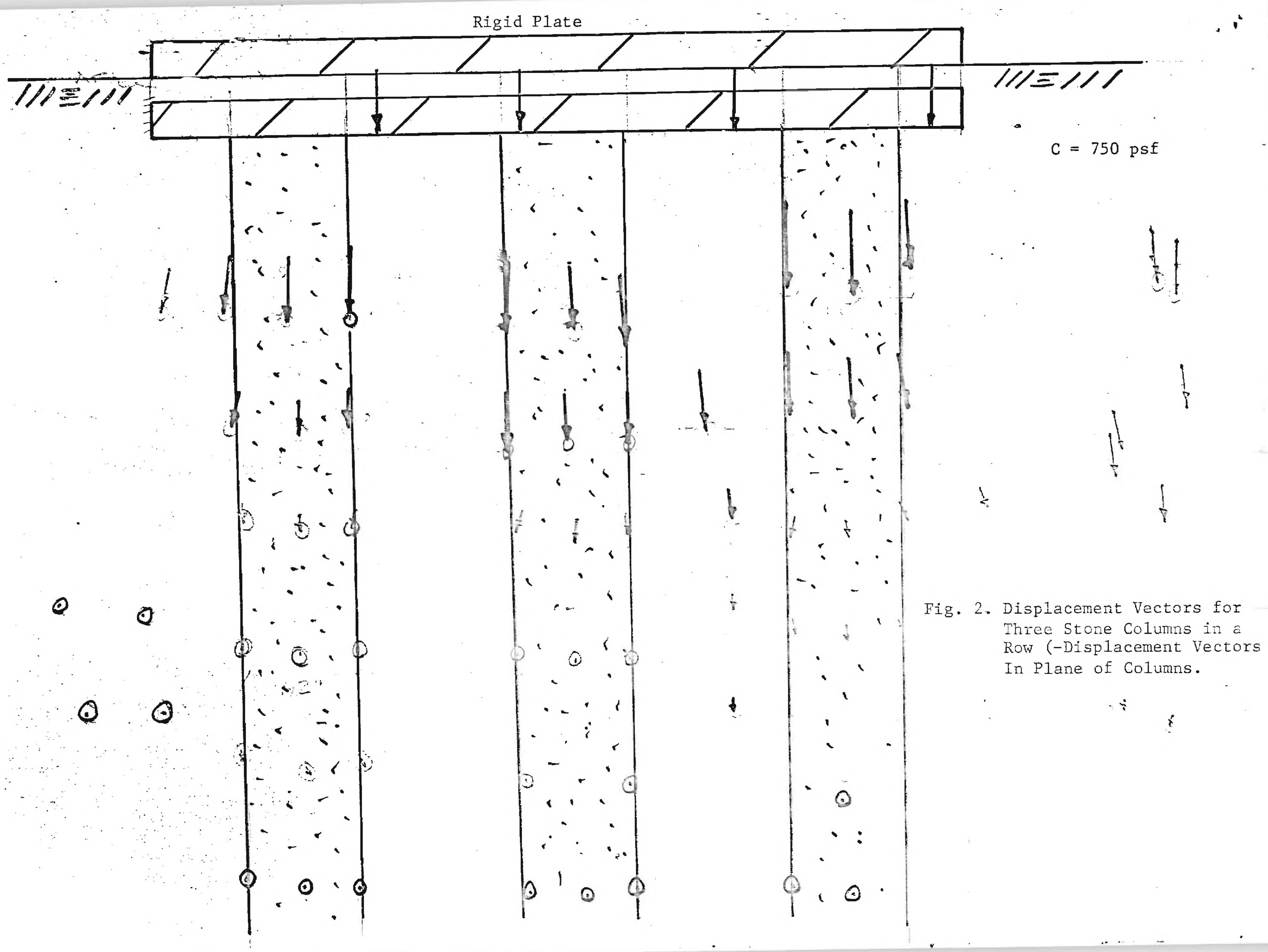


Fig. 2. Displacement Vectors for Three Stone Columns in a Row (-Displacement Vectors In Plane of Columns.

E-20-686

MONTHLY PROGRESS REPORT FEDERAL HIGHWAY ADMINISTRATION	Date of Report April 8, 1982

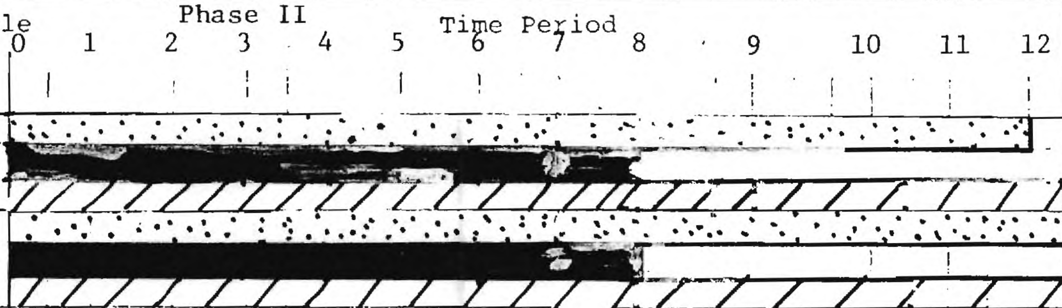
1 Project No.	2 Project Title	3 Report No. 18
DTFH61-80-C-00111 GA TECH· E20-686	Development of Methodology for the Design and Construction of Stone Columns	From Feb. 1, 1981 To Feb. 28, 1981

4 Research Agency	5 Project Director(s)
Georgia Institute of Technology	Richard D. Barksdale, Director Robert C. Bachus, Co-Director

6 Starting Date	7 Completion Date	8 % Time Expended	9 Schedule Status	10 Sufficiency of Funds
Sept. 1, 1980	Phase I & II Aug. 31, 1982	57%	☐ Ahead ☒ Behind ☐ On	☒ Sufficient ☐ Insufficient

Funds Authorized		Funds Expended			
11 Total (Phase I & II)		13 Total to Date	%		15 Report Month
FHWA $97,800 GaTech $18,300		FHWA: $75,161 GaTech: $9,702	77 (53)		$3,024 (Ga.Tech: $0)

16 Project Schedule — Research Tasks — Phase II — Time Period



Research Tasks	% Task Completed
TASK A Lab Study	50
TASK B Design Methodology	75
Overall % Completed	65

Approved Schedule — Work Completed Schedule — Projected Completion Schedule

17 Progress This Month (By Task) (Refer to ATTACHMENT NO. 1 for Item 17)

(For Time Expended Refer to Table 1)

18 Work Planned for Next Month

Task A. Continue mass production of design data using 24 in. dia. circular tanks.

Task B. Finish nonlinear analysis of unit cell model to establish the effect of lateral spreading on settlement.

19 Significant Technical Information, Recommendations, Implementation

Refer to 17 above.

20 Problems

Refer to 17 above.

21 Report Prepared by

	Dr. R. D. Barksdale	Project Director
Signature	Name	Title

Table 1
Summary of Man-Hours Expended

	Total	February	% By Task Task (Feb. Work) A	B
R. D. Barksdale, Project Director	1491	45	40%	60%
R. C. Bachus, Co-Director	960	25	100%	0
Roger Blackwell, Graduate Student	816	24	0	100%
Brent Reid, Student	121	9.5	100%	0
George Kaffezakis, Graduate Student	310	60	100%	0
Steve Long, Machinist	85	15	100%	0
Total	3783	163		

ATTACHMENT I

I. TASK A: Model Tests

The problem of getting good x-rays for the unit cell model was worked out. Two unit cell tests were performed. Vertical and lateral pressure cells have been installed in the rectangular box. A sample is ready to begin consolidating in the box.

Testing has begun in the large tank. Vibrating the sand was found to be better than ramming to form a model stone column.

II. TASK B: DESIGN METHODOLOGY

Computer work using the unit cell progressed although the analysis with "soft" boundaries was not complete.

The theory was completed for the local bearing failure analysis of stone columns. Attachment II summarizes this work. At present a computer program has been written and is being debugged to generate design curves for selected conditions.

ATTACHMENT II

LOCAL BEARING FAILURE OF STONE COLUMN (PRELIMINARY)

Stone columns are an effective method for resisting rotational shear failures involving soft clays in embankment and slopes [1]. For a conventional slope stability analysis the resisting shear force F developed by the stone column is determined by multiplying the effective normal force, W_N acting on the shear surface by the angle of internal friction of the stone, ϕ_s. The shear capacity, F, of the stone column can, under unfavorable conditions, be limited by a local bearing failure of the soft clay behind the stone column.

If the shear force in the stone column is sufficiently large compared to the strength of the surrounding clay, a secondary failure surface develops in the stone column extending downward from the circular arc failure surface (Fig. 1). The resulting wedge of failed stone is bounded above by the circular arc failure surface. The lower failure surface develops within the stone at an angle resulting in the minimum resistance to sliding. The shear force, F, applied to the top causes the wedge to slide downward and laterally in the direction of movement of the unstable soil mass above. Sliding of the wedge of stone is resisted by friction resistance of the stone developed along the bottom of the wedge and the passive lateral resistance of the adjacent clay. If the passive resistance of the clay is not sufficient, the stone wedge undergoes a local bearing failure by punching into the clay.

If a local bearing failure of the clay occurs behind the stone column the capacity of the column is reduced to that determined by the secondary wedge failure. A local bearing failure of the clay behind the stone column has been observed by Goughnour [2] during a direct shear test performed in the field on a stone column.

Local Bearing Failure. The limiting shear force that can be applied if a bearing failure controls can be obtained by considering the equilibrium of the wedge shown in Fig. 1. This wedge together with the forces acting on it are illustrated in Fig. 2. The notation shown in this figure is used in the subsequent derivations and is as follows:

W_s = effective force of stone in the wedge

γ_s = effective (bouyant) unit weight of stone in wedge

P_H = ultimate lateral resistance of the clay acting on the wedge

N,T = normal and shear force, respectively, exerted on the bottom surface of the wedge

W_N, T = normal and shear force, respectively, exerted on the top surface of the wedge

R = radius of the stone column

D = diameter of the stone column

ϕ_s = angle of internal friction of the stone

α,β = angle of inclinations of the lower and upper surfaces of the wedge, respectively.

The upper surface of the wedge makes an angle β with the horizontal. This upper surface coincides with the circular arc failure surface (Fig. 1). The lower surface of the wedge makes an angle of α with the horizontal. Now consider equilibrium of the wedge. To develop the required relationship for F, first sum forces acting on the wedge in the vertical direction and solve for the unknown normal force N acting on the bottom of the wedge obtaining

$$N = \frac{W_s + W_N \cos\beta + F\sin\beta}{\cos\alpha + \tan\phi_s \sin\alpha} \tag{1}$$

where the forces and angles are shown in Fig. 2.

Now sum the forces acting on the wedge in the horizontal direction, substitute for the unknown force N using equation (1), and solve for the limiting force F obtaining

$$F = \frac{W_N(\sin\beta + \lambda\cos\beta) + \lambda W_s + P_H}{\cos\beta - \lambda\sin\beta} \tag{2}$$

where:

$$\lambda = \frac{\tan\phi_s\cos\alpha - \sin\alpha}{\cos\alpha + \tan\phi_s\sin\alpha}$$

$$W_s = \pi|\tan\alpha - \tan\beta|R^3\gamma_s$$

In the deviation of equation (2) the effect of the following factors were neglected: (1) adhesion between the stone column and clay, (2) effects of adjacent columns, and (3) the effect of outward lateral spreading of the stone columns. Neglecting adhesion and the effect of adjacent columns should introduce a factor of conservation in predicting the effect of a local bearing failure [3-6]. These effects are offset by neglecting lateral spreading which should be on the unconservative side.

Lateral Bearing Failure in Clay. The ultimate passive pressure developed by the clay as the wedge pushes against it can be calculated using the theory presented by Broms [3] for a single, laterally loaded pile embedded in frictionless clay. As shown in Fig. 3 the ultimate lateral pressure q_h at the surface is taken to be $q_h = 2c$ with the resistance increasing linearly to a depth of 3 pile diameters where it reaches a maximum limiting value of $q_h = 9c$. Near the surface the failure occurs due to the upward flow of clay toward the surface. With increasing depth the failure becomes one of the plastic flow of the clay from the front of the pile around the sides (Fig. 3).

For a single, rough pile having full cohesion, plastic theory [3,4] indicates below a depth of approximately 3 diameters the ultimate lateral capacity is about q_h = 11 to 12c. Use of an ultimate resistance of 9c is prudent and may even be slightly on the conservative side.

The ultimate strength of q_h = 9c presented by Broms is thus slightly on the conservative side compared with plastic flow theory. Use of q_h = 9c is further justified since this value is equal to the end bearing capacity of deep piles embedded in a cohesive soil. The value of q_h = 2c used at the surface is also realistic since it equals about 40% of the bearing capacity of the clay in the vertical direction.

Now consider the ultimate lateral pressure developed on a wedge of stone making an angle α and β with the horizontal as shown in Fig. 2. Using the pressure distribution shown in Fig. 3, the ultimate passive pressure developed in the clay for a depth $(h + z_o) \leq 3D$ is

$$P_H = \frac{14}{3} R\ c\ \psi\ [h + z_o + R\ (1.714 + \tan\alpha)] \tag{3}$$

and for a depth $h + z_o > 3D$.

$$P_H = 36R^2\ c\ \psi \tag{4}$$

where: R = radius of stone column

c = cohesion

ψ = $\tan\alpha - \tan\beta$

h = depth of fill above the stone column

z_o = depth of the circular arc failure surface below the top of the stone column

The sign convention used for α and β is shown in Fig. 4.

Once a trial circular arc failure surface has been selected, the value of β is known. The angle α is then determined to give the minimum value of shear force F applied to the top of the wedge to cause a bearing failure.

Calculation of Limiting Shear Force. The limiting shear force F in each column for a given circular arc sliding surface is calculated as follows:

1. Determine the angle β and calculate the effective normal force, W_N (Fig. 4) at the point on the stone column where the circular arc intersects the center of the stone column (Fig. 1).

2. Select at least three trial values of the angle of inclination α of the lower surface of the wedge.

3. For each value of α calculate the ultimate lateral soil resistance, P_H using equation (3) or (4) and a representative value of the undrained shear strength c of the clay.

4. For each value of α, calculate F for a bearing failure in the clay using equation (2).

5. Plot the shear force F obtained from equation (2) as a function of α and select the minimum value of F.

6. Calculate the shear force F that can act on the column if a local bearing failure does not develop: $F = W_N \tan\phi_s$.

7. If a local bearing failure of the clay controls the force calculated in Step 5 will be less than that calculated in Step 6. In the stability analysis use the smaller of these forces.

Design. The likelyhood of a local bearing failure increases as the shear strength of the clay decreases and as a greater angle of internal friction is used in design. For example, if an angle of internal friction, ϕ_s of the stone column of 42° is used, a local bearing failure can occur in cohesive soils having undrained shear strengths less than about 400 psf (19 kN/m^2). A local bearing failure can occur in higher strength cohesive soils if ϕ_s values greater than 42° are used in design. Therefore, when stability is being analyzed in saturated, very soft and soft clays or silts the effect on the overall slope stability must be considered of a local bearing failure.

A practical method is therefore needed of incorporating the concept of a local bearing failure into the design procedure. The local bearing failure mechanism can be easily introduced into a slope stability design using the concept of a limiting angle of internal friction ϕ_s of the stone. Before the stability analysis is performed, a value of ϕ_s is selected sufficiently low to preclude a local bearing type failure from occurring anywhere along the circle. Using this simplified approach several representative points are selected along the estimated failure circle(s). The effective normal stress, W_N and inclination of the failure circle β at the points is then determined. The shear force F based on a local bearing failure are calculated for each point and compared with the frictional force $F = W_N \tan\phi_s$. If a bearing failure controls, the value of ϕ_s is reduced and the process repeated until the force F determined from local bearing considerations approximately equals the frictional force.

The other alternatives is to introduce the procedure for evaluating the effect of local bearing directly into a computer program so that the correct value of F is always used for each stone column. Direct use of this method in a stability analysis computer porgram is desirable but would require modifying the program.

If care is exercised, satisfactory results can be obtained by limiting the value of the angle of internal friction of the stone, ϕ_s to prevent a bearing failure from occurring. After using the predetermined value of ϕ_s in a stability analysis, the results should be reviewed to determine if the assumed critical failure circle is sufficiently close to that used to estimate the limiting value of ϕ_s. If time permits, design charts will be developed for selecting safe ϕ_s values by using a computer program to solve equation (2).

REFERENCES

1. DiMaggio, J. A., STONE COLUMNS FOR HIGHWAY CONSTRUCTION, DEMONSTRATION PROJECT NO. 46, Federal Highway Administration, Region 15, Demo. Projects Division, Arlington, Virginia, June 1978.

2. Goughnour, R. R., PERSONAL COMMUNICATION, September, 1981.

3. Broms, B. B., Lateral Resistance of Piles in Cohesive Soils, PROCEEDINGS, ASCE, Vol. 90, No. SM2, March, 1964, p.21-62.

4. Poulos, H. G., and Davis, E. H., PILE FOUNDATION ANALYSIS AND DESIGN, John Wiley and Sons, New York, 1980, pp. 145-148.

5. Meyerhof, G. G., and Chaplin, M. A., "The Compression and Bearing Capacity of Cohesive Soils", British Journal of Applied Physics, Vol. 4, Jan., 1953, pp. 20-26.

6. Winterkorn, H. F., and Fang, H. Y., FOUNDATION ENGINEERING HANDBOOK, Van Nostrand Reinhold Co., New York, 1975, p. 139-141.

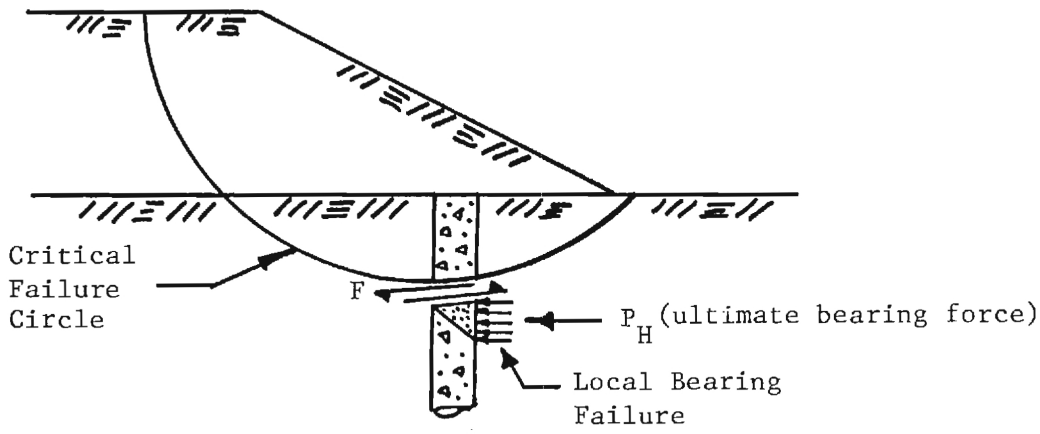


FIGURE 1. WEDGE TYPE LOCAL BEARING FAILURE OF A STONE COLUMN.

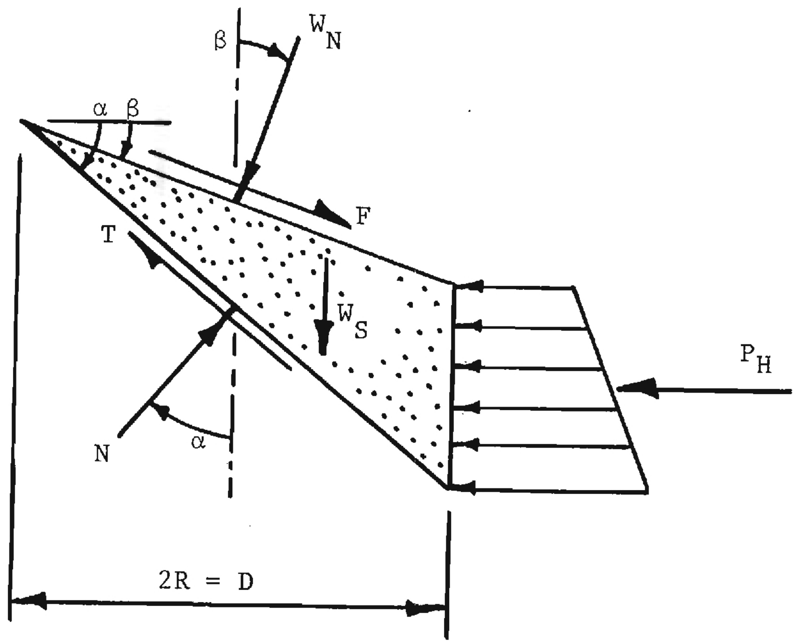


FIGURE 2. LOCAL BEARING CAPACITY FAILURE WEDGE IN STONE COLUMN.

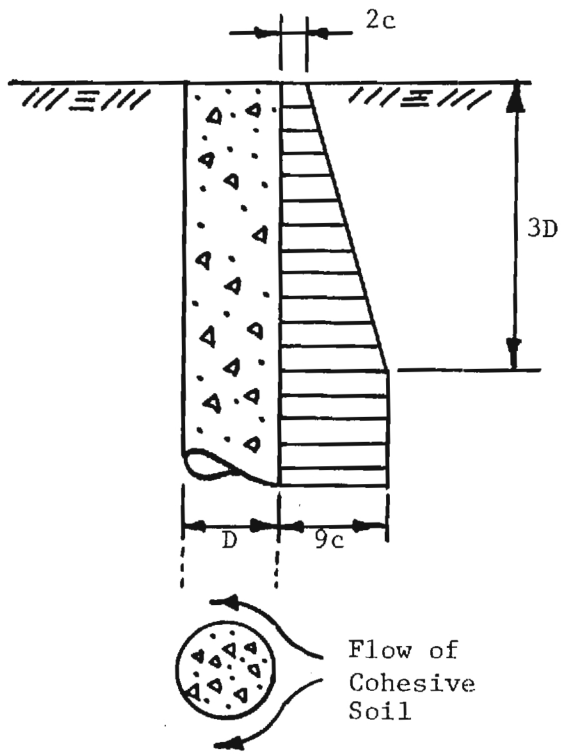
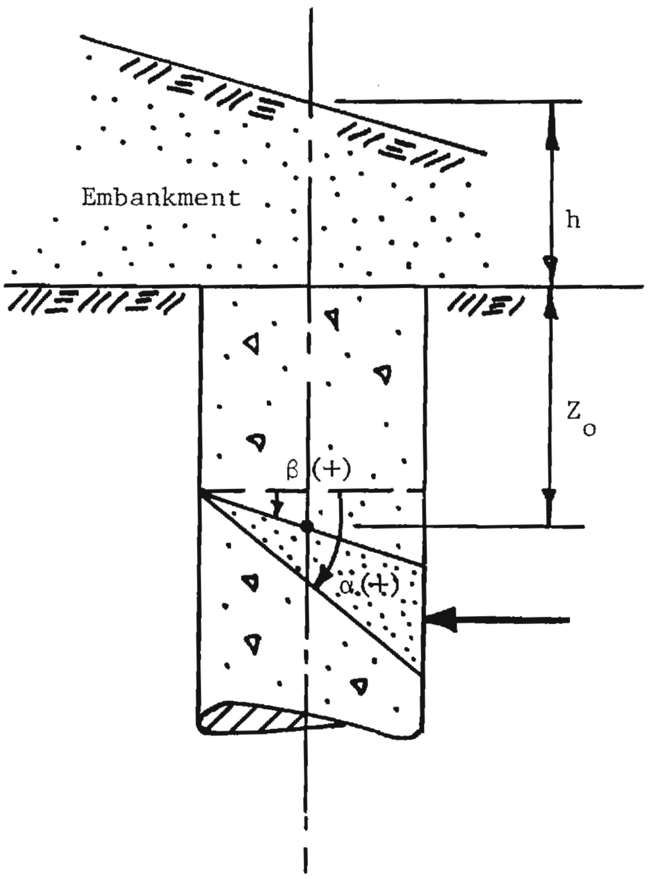
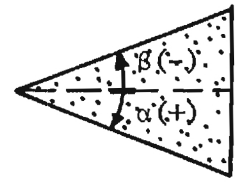


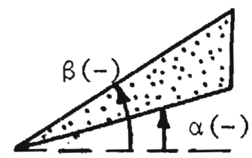
FIGURE 3. BEARING CAPACITY OF A RIGID PILE TRANSLATING LATERALLY IN A COHESIVE SOIL.



(a) Embedded Column - α and β Positive



(c) α Positive and β Negative



(b) α and β Negative

FIGURE 4. NOTATION USED IN FORMULAS FOR LOCAL BEARING FAILURE OF A STONE COLUMN.

MONTHLY PROGRESS REPORT FEDERAL HIGHWAY ADMINISTRATION	Date of Report May 20, 1982

1 Project No.	2 Project Title	3 Report No. 19
DTFH61-80-C-00111 GA TECH· E20-686	Development of Methodology for the Design and Construction of Stone Columns	From March 1, 1982 To March 31, 1982

4 Research Agency	5 Project Director(s)
Georgia Institute of Technology	Richard D. Barksdale, Director Robert C. Bachus, Co-Director

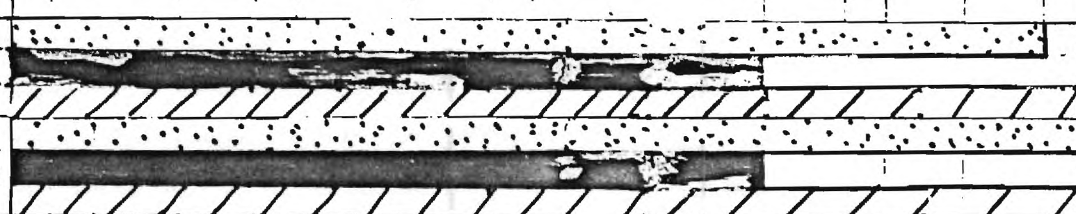
6 Starting Date	7 Completion Date	8 % Time Expended	9 Schedule Status	10 Sufficiency of Funds
Sept. 1, 1980	Phase I & II Aug. 31, 1982	79%	☐ Ahead ☒ Behind ☐ On	☒ Sufficient ☐ Insufficient

Funds Authorized		Funds Expended			
11 Total (Phase I & II)		13 Total to Date	%		15 Report Month
FHWA $97,800 GaTech $18,300		FHWA: $82,811 GaTech: $9,702	85 (53)		$3,446 (Ga.Tech: $0)

16 Project Schedule

Phase II

Research Tasks	Time Period 0 1 2 3 4 5 6 7 8 9 10 11 12	% Task Completed
TASK A Lab Study		60
TASK B Design Methodology		80
Overall % Completed		70



Approved Schedule — Work Completed Schedule — Projected Completion Schedule

17 Progress This Month (By Task) (Refer to ATTACHMENT NO. 1 for Item 17)

(For Time Expended Refer to Table 1)

18 Work Planned for Next Month

Task A. Continue mass production of design data using 24 in. dia. circular tanks.

Task B. Continue nonlinear analysis of unit cell model with slip and soft boundary to establish the effect of lateral spreading and slip on settlement.

19 Significant Technical Information, Recommendations, Implementation

Refer to 17 above.

20 Problems

Refer to 17 above.

21 Report Prepared by

Signature	Name	Title
	Dr. R. D. Barksdale	Project Director

Table 1
Summary of Man-Hours Expended

	Total	March	% By Task Task (Feb. Work) A	B
R.D. Barksdale, Project Director	1511	20	40%	60%
R. C. Bachus, Co-Director	979	19	100%	0
Roger Blackwell, Graduate Student*	836	20	0	100%
Brent Reid, Student	130	9	100%	0
George Kaffazakis, Graduate Student	370	60	100%	0
Steve Long, Machinist	95	10*	100%	0
E. Bradley, Student	26.5	26.5	100%	0
Jose' Perez, Graduate Student*	45	45	0	100%
Total	3992			

* No cost to project

ATTACHMENT I

I. TASK A - MODEL STUDIES

During March, primary emphasis was placed on the consolidation and testing of the unit cell. Tests were conducted at 0, 7, 25 and 100% sand replacement. All tests involving clay were consolidated from a slurry, as has been the standard procedure. Lead shot was placed in selected specimens to monitor movement and verify the one-dimensional consolidation of the clay/sand combination. These tests will be used in comparison with the long-term model test studies conducted by the Building Research Establishment, England, as well as our own analytical work. Results indicate that small replacement ratios have little effect on the resulting deformation. Replacement ratios of ~25% realize approximately 40 - 50% reduction of settlement relative to tests on unimproved clay.

Preliminary work is also underway on using the 0.75 in. diameter pressure cells in the top cap of the unit cell to measure the amount of stress concentration during the loading. Based on these results top caps for the unit cell as well as the rectangular box will be designed and constructed.

Five tests to failure were conducted in the 24 in. diameter tank to establish relationships between the number of sand columns and bearing capacity.

II. TASK B - DESIGN METHODOLOGY

The finite element computer work using the unit cell with "soft" boundaries was completed. Slip elements are being added to allow considering both slip and a soft boundary. To get realistic settlement estimates apparently everything possible that causes settlement must be included in the analysis.

The computer program has been written and debugged to generate design curves for selected conditions for local bearing failure.

E-20-686

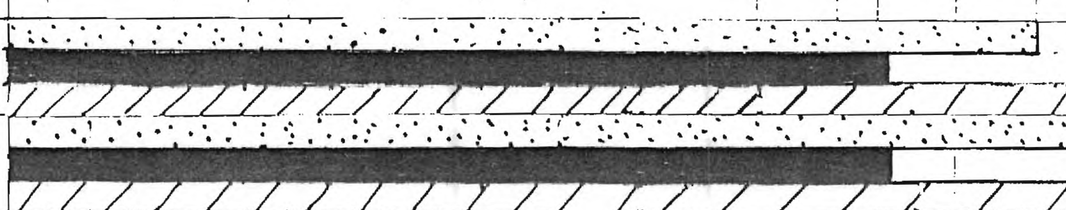
MONTHLY PROGRESS REPORT FEDERAL HIGHWAY ADMINISTRATION			Date of Report June 8, 1982
1 Project No. DTFH61-80-C-00111 GA TECH· E20-686	2 Project Title Development of Methodology for the Design and Construction of Stone Columns		3 Report No. 20 From April 1, 1982 To April 30, 1982
4 Research Agency Georgia Institute of Technology		5 Project Director(s) Richard D. Barksdale, Director Robert C. Bachus, Co-Director	

6 Starting Date	7 Completion Date	8 % Time Expended	9 Schedule Status	10 Sufficiency of Funds
Sept. 1, 1980	Phase I & II Aug. 31, 1982	83%	☐ Ahead ☒ Behind ☐ On	☒ Sufficient ☐ Insufficient

Funds Authorized	Funds Expended		
11 Total (Phase I & II)	13 Total to Date	%	15 Report Month
FHWA $97,800 GaTech $18,300	FHWA· $84,951 GaTech: $9,702	87 (53)	$2,141 (Ga.Tech: $0)

16 Project Schedule — Phase II — Time Period

Research Tasks	Time Period 0 1 2 3 4 5 6 7 8 9 10 11 12	% Task Completed
TASK A Lab Study		65
TASK B Design Methodology		86
Overall % Completed		80



Approved Schedule — Work Completed Schedule — Projected Completion Schedule

7 Progress This Month (By Task) (Refer to ATTACHMENT NO. 1 for Item 17)

Note: 1. Refer to ATTACHMENT NO. 1 for Item 17.
2. For Time Expended Refer to Table 1.
3. For Project Cost by Task Refer to Table 2.

8 Work Planned for Next Month

Task A. Continue mass production of design data using 24 in. dia. circular tanks.

Task B. Develop influence charts for unit cell analysis with slip and soft boundary. Investigate use of standard slope stability programs considering stress concentration.

9 Significant Technical Information, Recommendations, Implementation

Refer to 17 above.

0 Problems

Refer to 17 above.

1 Report Prepared by

	Dr. R. D. Barksdale	Project Director
Signature	Name	Title

Table 1
Summary of Man-Hours Expended

	Total	April	% By Task Task (Feb. Work) A	B
R.D. Barksdale, Project Director*	1536	25	40%	60%
R.C. Bachus, Co-Director	1001	22	100%	0
Roger Blackwell, Graduate Student*	854	18	0	100%
Brent Reid, Student	147	16.5	100%	0
George Kaffazakis, Graduate Student	430	60	100%	0
Steve Long, Machinist*	110	15	100%	0
E. Bradley, Student	60	33	100%	0
Jose' Perez, Graduate Student*	85	40	0	100%

Total - 4223

* No cost to project.

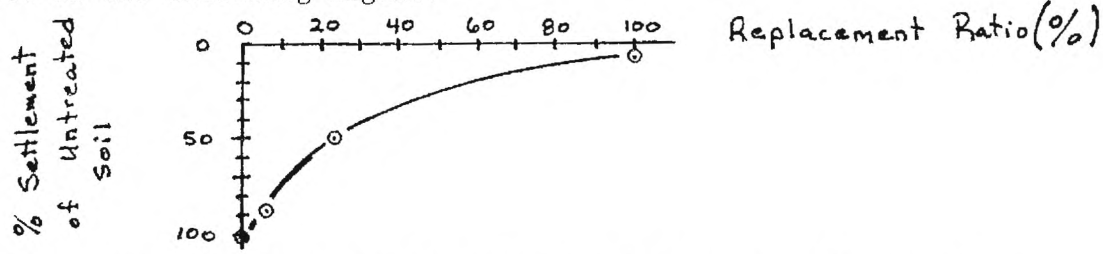
Table 2
Project Cost by Task

Task A (Phase II)	$1,856
Task B (Phase II)	$ 285

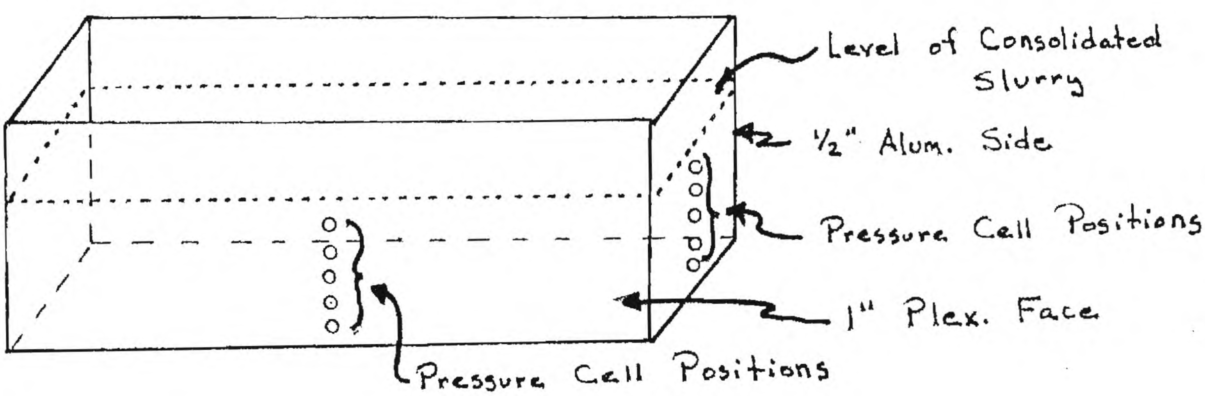
ATTACHMENT I

I. TASK A - MODEL STUDIES

In April, the model tests on the unit cell continued as pressure cells were incorporated into the top cap. One pressure cell was placed at the center of the cap while a second was moved nearer the caps outer edge. This arrangement will allow for the independent measurement of stresses within the clay and the column. The pressure cells have undergone minor changes to facilitate construction and calibration. Problems were detected under prolonged exposure to moisture, due to condensation on the cell walls. Additionally, improvements of the electrical connection within the cell were explored. Calibration of the cells indicates an extremely accurate and consistent reflection of the stress acting on the membrane. The initial test results which were discussed last month, were carefully evaluated and the results indicate that only about 50% of the consolidation settlement anticipated in the unimproved soft clay soil is realized when a replacement ratio of 25% is selected. Details of these results are summarized on the following figure.



The pressure cells used for the top cap of the unit cell are also used in the rectangular box. Unlike the unit cell, however, the cells are replaced in the side and face of the rectangular box as well as in the loading cap. The details of placement are shown in the accompanying sketch.



Note that all the pressure cells are rigidly attached to stationary walls. Five placements are prescribed at various depths with the soil to investigate the relationship between pressure distribution and depth. Dummy cells are also provided to plug the unused holes during testing. Placement and testing of the stone column groups will take advantage of the instrumented wall and face and treat them as planes of symmetry. In this manner the stresses developed within stone column groups may be evaluated. The top cap for each group test will also have provisions for pressure cells placed atop and between the stone columns.

The tank studies are presently being used to develop the relationship between bearing capacity of a single stone column and reasonably small groups of stone columns loaded by a rigid plate. These charts should be completed by the end of next month.

II. TASK B - DESIGN METHODOLOGY

The finite element computer data deck was generated using a soft boundary and slip elements. Design curves will be generated next month. About 41 computer runs were made to generate design curves for selected conditions of local bearing. Design Charts will be plotted next month.

MONTHLY PROGRESS REPORT

FEDERAL HIGHWAY ADMINISTRATION

Date of Report: July, 16, 1982

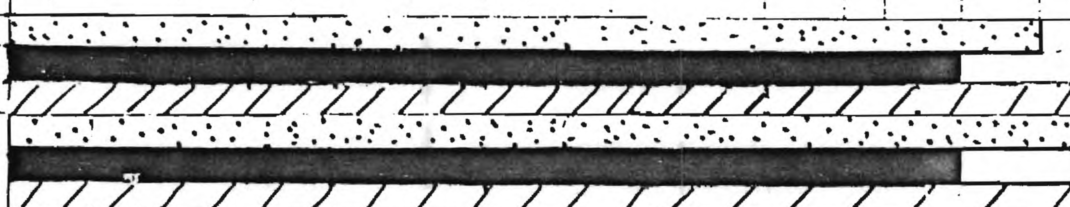
1 Project No.	2 Project Title	3 Report No. 21
DTFH61-80-C-00111 GA TECH · E20-686	Development of Methodology for the Design and Construction of Stone Columns	From May 1, 1982 To May 31, 1982

4 Research Agency	5 Project Director(s)
Georgia Institute of Technology	Richard D. Barksdale, Director Robert C. Bachus, Co-Director

6 Starting Date	7 Completion Date	8 % Time Expended	9 Schedule Status	10 Sufficiency of Funds
Sept. 1, 1980	Phase I & II Aug. 31, 1982	88%	☐ Ahead ☒ Behind ☐ On	☒ Sufficient ☐ Insufficient

Funds Authorized	Funds Expended		
11 Total (Phase I & II)	13 Total to Date	%	15 Report Month
FHWA $97,800 GaTech $18,300	FHWA · $87,781 GaTech: $9,702	87 (53)	$ 2,830 (Ga.Tech: $0)

16 Project Schedule — Research Tasks — Phase II — Time Period 0 1 2 3 4 5 6 7 8 9 10 11 12 — % Task Completed



Research Tasks	% Task Completed
TASK A Lab Study	75
TASK B Design Methodology	92
Overall % Completed	85

Approved Schedule — Work Completed Schedule — Projected Completion Schedule

17 Progress This Month (By Task) (Refer to ATTACHMENT NO. 1 for Item 17)

Note: 1. Refer to ATTACHMENT NO. 1 for Item 17.
2. For Time Expended Refer to Table 1.
3. For Project Cost by Task Refer to Table 2.

18 Work Planned for Next Month

Task A. Continue mass production of design data using 24 in. dia. circular tanks.

Task B. Continue developing influence charts for unit cell analysis with slip & soft boundary. Investigate use of standard slope stability programs considering stress concentration.

19 Significant Technical Information, Recommendations, Implementation

Refer to 17 above.

20 Problems

Refer to 17 above.

21 Report Prepared by

	Dr. R. D. Barksdale	Project Director
Signature	Name	Title

ATTACHMENT I

I. TASK A - MODEL STUDIES

Model tests on the unit cell were continued with primary emphasis placed on (a) vertical stress distribution at the surface of the clay and sand column, and (b) the longitudinal distribution of lateral stress. The pressure cells which were originally designed for this project were determined to be ineffective in the present application. The major problem concerned condensation within the cell which restricted movement of the central plunger. Numerous attempts to revise the design resulted in similar problems. Therefore, a more rugged and reliable pressure cell was designed and fabricated. The new design incorporates a thin exposed metal diaphram and an internally mounted diaphram strain gage. This concept offers many advantages over the original design. The cell can more effectively be sealed against moisture than the exposed rubber membrane. It monitors the stresses continuously and does not require a balancing air supply. Disphrams of varying stiffness can be used depending on the anticipated load level or sensitivity. The cells are connected through a 10 channel switching and balancing box to a central readout unit. The results are very promising at this time. The results of one test, UC-6-0.07P, is included in this progress report as Figure 1, which presents the variation of stress concentration with elapsed time of loading for 3 load increments. Additional cells are presently being fit into the side of the unit cell and the large rectangular box for additional testing.

II. TASK B - DESIGN METHODOLOGY

Local bearing design curves were plotted; a bearing capacity theory for predicting stone column group performance was developed. Development of the design curves for settlement did not make much progress this month. In general the theory portion, considering the intended level of effort to go into it, has progressed nicely.

Table 1
Summary of Man-Hours Expended

	Total	May	% By Task Task A	B
R.D. Barksdale, Project Director*	1701	165	40%	60%
R. C. Bachus, Co-Director	1026	25	100%	0
Roger Blackwell, Graduate Student*	873	19	0	100%
Brent Reid, Student	147	-	-	0
George Kaffazakis, Graduate Student	490	60	100%	0
Steve Long, Machinist*	140	30	100%	0
E. Bradley, Student	87	27	100%	0
Jose' Perez, Graduate Student*	117	32	0	100%
Bahrt, Steve	60	30	-	-

Total - 4641

* No cost to project this month.

Table 2
Project Cost by Task

Task A (Phase II)	$2,590
Task B (Phase II)	$ 240

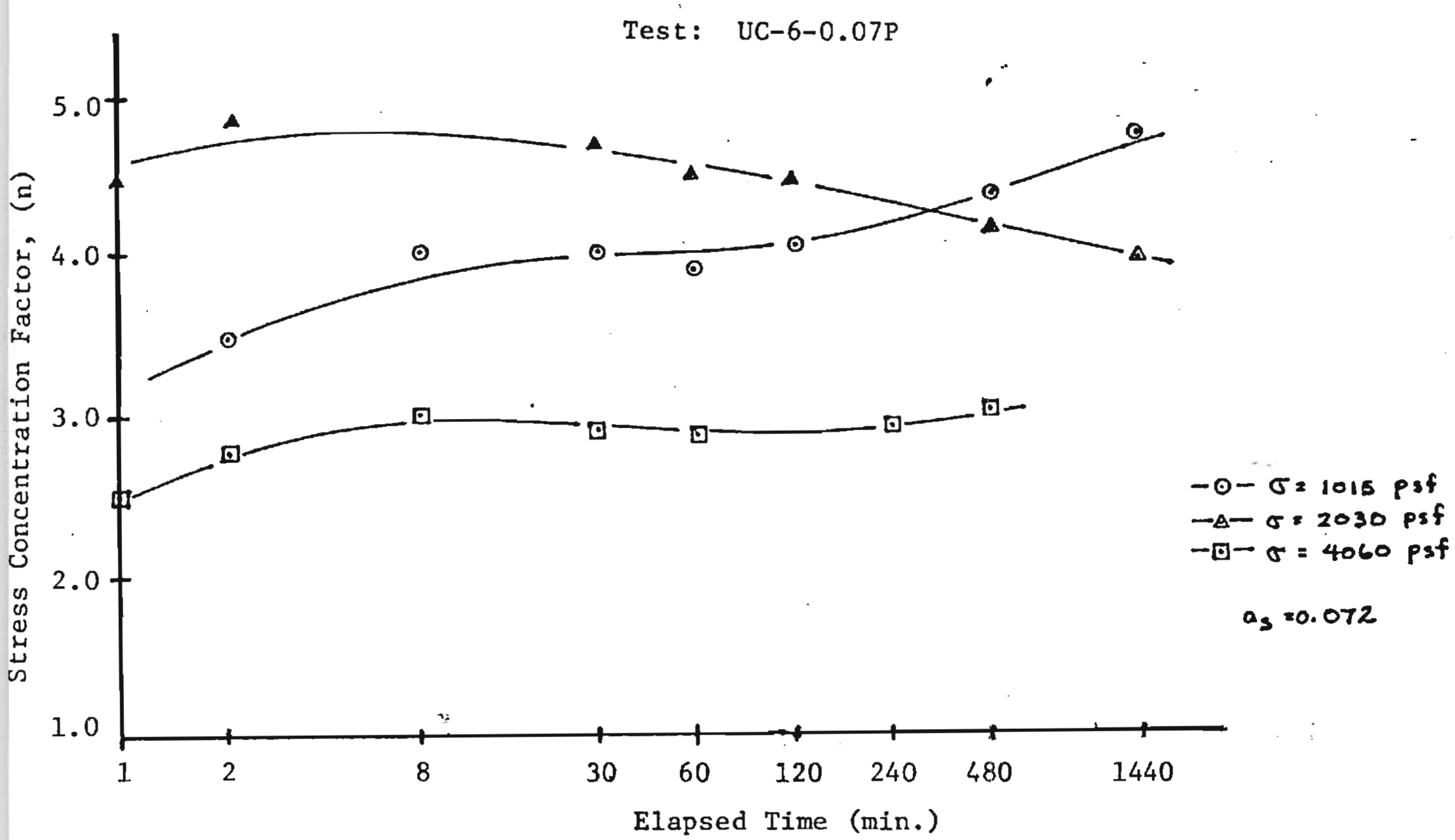


Figure 1. Variation of Stress Concentration Factor with Elapsed Time of Testing for 3 Load Levels.

E-20-686

MONTHLY PROGRESS REPORT

FEDERAL HIGHWAY ADMINISTRATION

Date of Report: July 26, 1982

1 Project No.	2 Project Title	3 Report No. 22
DTFH61-80-C-00111 GA TECH · E20-686	Development of Methodology for the Design and Construction of Stone Columns	From June 1, 1982 To June 30, 1982

4 Research Agency	5 Project Director(s)
Georgia Institute of Technology	Richard D. Barksdale, Director Robert C. Bachus, Co-Director

6 Starting Date	7 Completion Date	8 % Time Expended	9 Schedule Status	10 Sufficiency of Funds
Sept. 1, 1980	Phase I & II Aug. 31, 1982	93%	☐ Ahead ☒ Behind ☐ On	☒ Sufficient ☐ Insufficient

Funds Authorized / Funds Expended

11 Total (Phase I & II)	13 Total to Date	%	14. Balance	15 Report Month
FHWA $97,800 GaTech $18,300	FHWA · $89,474 GaTech: $9,702	87 (53)	FHWA: $8,326 GaTech: $8,598	$ 1,692 (Ga.Tech: $0)

16 Project Schedule

Phase II — Time Period 0 1 2 3 4 5 6 7 8 9 10 11 12

Research Tasks	% Task Completed
TASK A Lab Study	80
TASK B Design Methodology	95
Overall % Completed	88

Approved Schedule · Work Completed Schedule · Projected Completion Schedule

7 Progress This Month (By Task)

Note: 1. Refer to ATTACHMENT NO. 1 for Item 17.
2. For Time Expended Refer to Table 1.
3. For Project Cost by Task Refer to Table 2.

Work Planned for Next Month

Task A. Continue mass production of design data using 24 in. dia. circular tanks. Continue instrumented model tests.

Task B. Continue developing influence charts for unit cell analysis with slip & soft boundary.

Significant Technical Information, Recommendations, Implementation.

Refer to 17 above.

Problems

Refer to 17 above.

Report Prepared by

	Dr. R. D. Barksdale	Project Director
Signature	Name	Title

Table 1
Summary of Man-Hours Expended

	Total	June	% By Task Task A	B
R.D. Barksdale, Project Director*	1896	195	10%	90%
R. C. Bachus, Co-Director	1066	40	100%	0
Roger Blackwell, Graduate Student*	885	12	0	100%
Brent Reid, Student	147	-	-	0
George Kaffazakis, Graduate Student	550	60	100%	0
Steve Long, Machinist*	164	24	100%	0
E. Bradley, Student	96	8	100%	0
Jose' Perez, Graduate Student*	152	35	0	100%
Bahrt, Steve	99	39	-	-
	Total - 5055			

* No cost to project this month.

Table 2
Project Cost by Task

Task A (Phase II)	$1622
Task B (Phase II)	$ 70

ATTACHMENT I

DEVELOPMENT OF BEARING CAPACITY ANALYSIS FOR STONE COLUMN GROUPS

DEVELOPMENT OF BEARING CAPACITY ANALYSIS FOR STONE COLUMN GROUPS

Consider the ultimate strength of either a square or infinitely long, rigid concrete footing resting on the surface of a cohesive soil reinforced with stone columns as illustrated in Fig. 16. Assume the foundation is loaded quickly so that the undrained shear strength is developed in the cohesive soil, with the angle of internal friction being negligible. Also neglect cohesion in the stone column. Finally, assume the full shear strength of both the stone column and cohesive soil is mobilized. The ultimate bearing capacity of the group can now be determined by approximating the failure surface by two straight rupture lines. Such a theory was first developed for homogeneous soils by Bell and modified by Terzaghi and Sowers [1]. This theory compares favorably with the Bell bearing capacity theory and gives results reasonably close to the Terzaghi local bearing failure theory for homogeneous soils.

Assume as an approximation that the soil immediately beneath the foundation fails on a straight rupture surface, forming a triangular block as shown in Fig. 16. The average shear resistance of the composite soil would be developed on the failure surface. The ultimate stress q_{ult} that the composite soil can withstand is dependent upon the lateral, ultimate resistance σ_3 of the block to movement and the composite shear resistance developed along the inclined shear surface. From a consideration of equilibrium of the block the average shear strength parameters within the block are

$$[\tan\phi]_{avg} = \mu_s a_s \tan\phi_s \tag{16a}$$

$$c_{avg} = (1 - a_s)\, c \tag{16b}$$

where $[\tan\phi]_{avg}$ is the tangent of the composite angle of internal friction and c_{avg} is the composite cohesion on the shear surface beneath the foundation; a_s is the area replacement ratio, equation (3) and μ_s is the stress concentration factor for the stone, equation (8b).

The failure surface makes an angle α with the foundation, where α for the composite soil is

$$\alpha = 45 + \frac{\phi_{avg}}{2} \tag{17}$$

where $$\phi_{avg} = \tan^{-1}(\mu_s a_s \tan\phi_s).$$

To calculate the ultimate capacity for a group first determine the ultimate lateral pressure σ_3. For an infinitely long footing from classical earth pressure theory for a saturated clay having only cohesion c:

$$\sigma_3 = \frac{\gamma_c \ B \tan\alpha}{2} + 2c \tag{18}$$

where: σ_3 = average lateral confining pressure

γ_c = saturated or wet unit weight of the cohesive soil

B = foundation width

α = inclination of the failure surface as given by Equation (17)

c = undrained shear strength within the unreinforced cohesive soil

The lateral confining effect for a square foundation can be determined using the cavity expansion theory of Vesic, Equation (12). The Vesic cylindrical expansion theory gives the ultimate stress that can be exerted on the failure block by the surrounding soil. The three-dimensional failure on a cylindrical surface should give a satisfactory approximation of the three-dimensional failure of a square foundation.

Assuming the ultimate vertical stress q_{ult} (which is also assumed to be σ_1) and ultimate lateral stress σ_3 to be principle stresses, equilibrium of the wedge requires

$$q_{ult} = \sigma_3 \tan^2\alpha + 2c_{avg} \tan\alpha \tag{19}$$

where σ_3 is obtained from equation (18) and the other terms have been previously defined. The effect of soil weight within the wedge was conservatively neglected. The soil weight within the wedge would increase the composite shear resistance and could be included in the analysis by appropriately modifying equation (16a). Such a degree of refinement is not considered justified at this time.

The proposed method for estimating the ultimate capacity of stone column groups considers (1) foundation shape, (2) foundation size, (3) the angle of internal friction of the stone column, composite shear strength of the stone column reinforced soil, (4) the shear strength and overburden pressure in the soil surrounding the foundation, and (5) the compressibility of the surrounding soil as defined by the Rigidity Index, Equation (13). In applying this approach it must be remembered that a composite strength of the entire soil mass is considered mobilized; therefore for soft soils use of a reduced composite strength (less than the combined individual strengths of the two materials at failure may be required to reflect the actual shear resistance mobilized along the failure wedge.

Discussion. As one bound, a large stone column groups can be approximated as an infinitely large group of columns. A stone column and its tributary soil located on the interior of the infinite array can be theoretically modeled using the unit cell concept. Since within a large group of stone columns the settlement of the soil and stone column is approximately equal , a rigid plate loading on the top of the unit cell can, as an approximation, be used. The model of a unit cell loaded by a rigid plate is quite similar to a one-dimensional consolidation test in which a bearing capacity failure does not occur since loading is along the K_o stress path line. Indeed, consolidation tests performed on unit cell models as a part of this study and also large scale tests at the Building Research Institute in England both showed similar performance to a consolidation test with failure not occurring. For most arrays of stone columns, it is not likely that the unit cell condition of boundary rigidity would ever be developed due to lateral spreading.

In practical applications due to lateral deformations of the stone column in the direction of least lateral resistance (Fig. 17), lateral consolidation of the soil surrounding the stone column, and the presence of locally very soft zones, the ultimate load capacity for most field conditions is limited to a finite value generally larger than for a single column. Fig. 18 shows the relationship developed between bearing capacity and number of columns for small stone column groups subjected to a rigid plate loading.

REFERENCES

1. Sowers, G. F., INTRODUCTORY SOIL MECHANICS AND FOUNDATIONS, MacMillian Publishing Co.

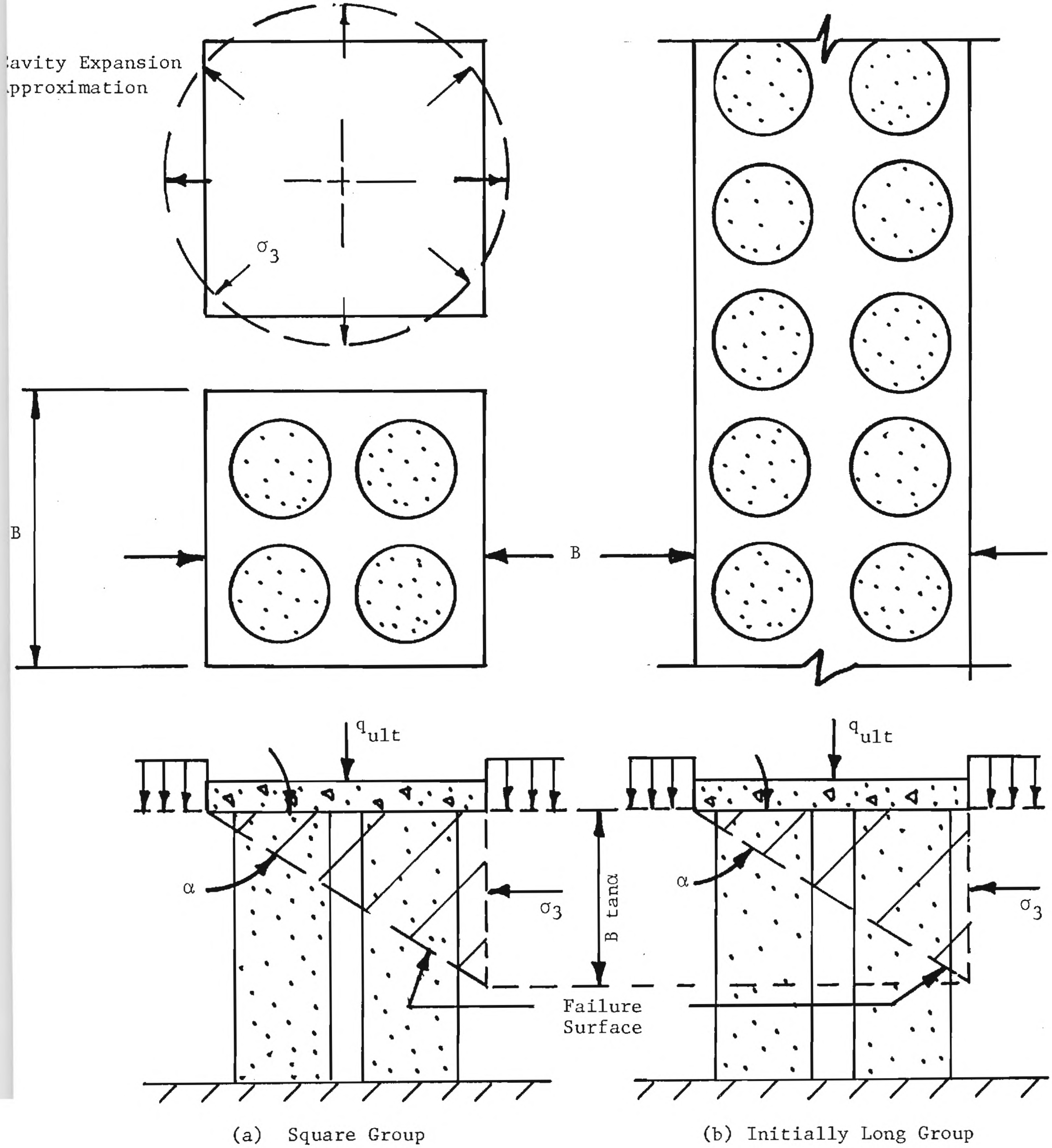


FIGURE 16. STONE COLUMN GROUP ANALYSIS.

MONTHLY PROGRESS REPORT

FEDERAL HIGHWAY ADMINISTRATION

Date of Report: August 20, 1982

1 Project No.	2 Project Title	3 Report No. 23
DTFH61-80-C-00111 GA TECH E20-686	Development of Methodology for the Design and Construction of Stone Columns	From July 1, 1982 To July 31, 1982

4 Research Agency	5 Project Director(s)
Georgia Institute of Technology	Richard D. Barksdale, Director Robert C. Bachus, Co-Director

6 Starting Date	7 Completion Date	8 % Time Expended	9 Schedule Status	10 Sufficiency of Funds
Sept. 1, 1980	Phase I & II Aug. 31, 1982	98%	☐ Ahead ☒ Behind ☐ On	☒ Sufficient ☐ Insufficient

Funds Authorized	Funds Expended			
11 Total (Phase I & II)	13 Total to Date	%	14. Balance	15 Report Month
FHWA $97,800 GaTech $18,300	FHWA $93,868 GaTech ($16,232)	96 (89)	FHWA: $3,932 (GaTech: $2,068)	$4,395 (Ga.Tech: $2,952)

16 Project Schedule — Phase II — Time Period 0 1 2 3 4 5 6 7 8 9 10 11 12

Research Tasks	% Task Completed
TASK A Lab Study	85
TASK B Design Methodology	98
Overall % Completed	92

Approved Schedule — Work Completed Schedule — Projected Completion Schedule

7 Progress This Month (By Task)

Note:

1. For Time Expended Refer to Table 1.
2. For Project Cost by Task Refer to Table 2.
3. Significant progress was made in writing the report. Also approximately 12 laboratory tests were performed in the circular tank and on slurry consolidated samples in the circular and rectangular, instrumented boxes.

Work Planned for Next Month

Task A. Continue mass production of design data using 24 in. dia. circular tanks. Continue instrumented model tests.

Task B. Complete developing influence charts for unit cell analysis with slip & soft boundary.

Task A & B. Complete write-up of the Final Report (draft).

Significant Technical Information, Recommendations, Implementation

Refer to 17 above.

Problems

Refer to 17 above.

Report Prepared by

	Dr. R. D. Barksdale	Project Director
Signature	Name	Title

Table 1
Summary of Man-Hours Expended

	Total	July	% By Task Task A	B
R.D. Barksdale, Project Director	2086	190	10%	90%
R.C. Bachus, Co-Director	1141	75	100%	0
Roger Blackwell, Graduate Student*	905	20	0	100%
Brent Reid, Student	147	-	-	-
George Kaffazakis, Grad. Student	550	-	-	-
Steve Long, Machinist*	184	20	100%	0
E. Bradley, Student	96	8	-	-
Jose' Perez, Graduate Student*	193	41	100%	-
Bahrt, Steve	137	38	100%	-
Ken Thomas, Research Assistant	466	173	100%	-
Total -	5905			

* No Cost to project this month.

Table 2
Project Cost by Task

Task A (Phase II)	$4,358
Task B (Phase II)	$2,989

MONTHLY PROGRESS REPORT FEDERAL HIGHWAY ADMINISTRATION	Date of Report Sept. 20, 1982

1 Project No.	2 Project Title	3 Report No. 24
DTFH61-80-C-00111 GA TECH· E20-686	Development of Methodology for the Design and Construction of Stone Columns	From Aug. 1, 1982 To Aug. 31, 1982

4 Research Agency	5 Project Director(s)
Georgia Institute of Technology	Richard D. Barksdale, Director Robert C. Bachus, Co-Director

6 Starting Date	7 Completion Date	8 % Time Expended	9 Schedule Status	10 Sufficiency of Funds
Sep. 1, 1980	Phase I & II Aug. 31, 1982	100%	☐ Ahead ☒ Behind ☐ On	☒ Sufficient ☐ Insufficient

Funds Authorized	Funds Expended			
11 Total (Phase I & II)	13 Total to Date	%	14. Balance	15 Report Month
FHWA $97,800 G Tech $18,300	FHWA· $97 800 GaTech($18,300)	100 (100)	FHWA: 0 (GaTech: 0	$4,380 (Ga.Tech: $2,068)

16 Project Schedule — Phase II — Time Period

Research Tasks	0	1	2	3	4	5	6	7	8	9	10	11	12	% Task Completed
TASK A Lab Study														95
TASK B Design Methodology														100
Overall % Completed														98

Approved Schedule — Work Completed Schedule — Projected Completion Schedule

7 Progress This Month (By Task)

Note:

1. For Time Expended Refer to Table 1.
2. For Project Cost by Task Refer to Table 2.
3. The final report (draft) was essentially complete during the month of August. Model te ts (Task A) were continued.

8 Work Planned for Next Month

Significant Technical Information, Recommendations, Implementation

Problems

Report Prepared by

Signature	Name	Title
	Dr. R. D. Barksdale	Project Director

Table 1
Summary of Man-Hours Expended

	Total		% By Task Task A	B
R.D. Barksdale, Project Director	2227	141	15%	85%
R.C. Bachus, Co-Director	1206	65	50%	50%
Roger Blackwell, Graduate Student*	923	18	0	100%
Brent Reid, Student	147	-	-	-
George Kaffazakis, Grad. Student	550	-	-	-
Steve Long, Machinist*	209	25	100%	0
E. Bradley, Student	96	8	-	-
Jose' Perez, Graduate Student*	220	27	100%	-
Bahrt, Steve	137	38	100%	-
Ken Thomas, Research Assistant	639	173	90%	10
G. Callol, Student	30	30	0	100%
Total -	6384			

* No Cost to project this month.

Table 2
Project Cost by Task

Task A (Phase II)	$2,633	(637)*
Task B (Phase II)	$1,746	(3613)

* Ga Tech Matching Funds (part of matching funds for August were encumbered earlier).

E-20-686

Final Report

DESIGN AND CONSTRUCTION OF STONE COLUMNS

Prepared for

Federal Highway Administration
Office of Research
Materials Division
Washington, D.C. 20590

February, 1983

GEORGIA INSTITUTE OF TECHNOLOGY
A UNIT OF THE UNIVERSITY SYSTEM OF GEORGIA
SCHOOL OF CIVIL ENGINEERING
ATLANTA, GEORGIA 30332

Technical Report Documentation Page

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Design and Construction of Stone Columns	February 1983
	6. Performing Organization Code
7. Author(s) R. D. Barksdale and R. C. Bachus	8. Performing Organization Report No. SCEGIT-83-104 (E20-686)
9. Performing Organization Name and Address School of Civil Engineering Georgia Institute of Technology Atlanta, Georgia 30332	10. Work Unit No. (TRAIS) 11. Contract or Grant No. DTFH61-80-C-00111
12. Sponsoring Agency Name and Address Federal Highway Administration Office of Research Materials Division Washington, D.C. 20590	13. Type of Report and Period Covered Final August 1980-August 1982 14. Sponsoring Agency Code

15. Supplementary Notes

16. Abstract

Stone columns have been used since the 1950's as a technique for improving both cohesive soils and silty sands. Potential applications include (1) stabilizing the foundation soils to support embankments, approach fills and reconstruction work on weak cohesive soils, (2) supporting retaining structures (including Reinforced Earth), bridge bent and abutment structures on slightly marginal soft to stiff clays and loose silty sands, (3) landslide stabilization and (4) reducing liquefaction potential of clean sands. Also, stone columns under proper conditions can greatly decrease the time required for primary consolidation.

Stone columns are constructed using a long, cylindrical shaped vibrator inserted into the ground. In the wet process, which is most commonly employed, large quantities of water are used under high pressure. Stone, usually having a maximum size of 2 to 4 in. (51-102 mm), is added to the hole in 2 to 4 ft. (0.6-1.2 m) increments and repenetrated by the vibrator several times.

The report describes construction, field inspection, and design aspects of stone columns. Also, several case histories are described, and bearing capacity, settlement and stability design examples are given in the appendices.

17. Key Words	18. Distribution Statement
Construction, Ground Improvement, Design, Site Improvement, Field Performance, Specifications, Stone Columns	

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified	239	

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NOTATION

A = total area within the unit cell

A_c = area of cohesive soil within the unit cell

A_s = stone column area

a_h = design horizontal earthquake acceleration coefficient expressed as a fractional part of g

a_s = area replacement ratio, A_s/A

B = foundation width

C_c = virgin compression index of cohesive soil from one-dimensional consolidation test

C_α = coefficient of secondary compression, $C_\alpha = \Delta H/(H \log_{10} t_2/t_1)$

c = cohesion of soil

c_v = coefficient of consolidation in vertical direction (equation 27)

C_{v_r} = coefficient of consolidation in radial direction (equation 28)

D_e = equivalent diameter of unit cell (equations 1 and 2)

D = constructed diameter of stone column (Figs. 13 and 14)

$\tilde{D}$ = constrained modulus of elasticity, $\tilde{D} = E(1-\nu)/[(1+\nu)(1-2\nu)]$

$\tilde{D}_c$ = constrained modulus of elasticity of the tributary soil

$\tilde{D}_s$ = constrained modulus of elasticity of the stone

E = modulus of elasticity

E_b = modulus of elasticity of thin boundary around the unit cell used in nonlinear finite element analysis

E_c = modulus of elasticity of soil within the unit cell

E_s = modulus of elasticity of the stone column

e_o = initial void ratio of cohesive soil

F = shear force on upper failure surface in stone column undergoing local bearing failure (Appendix B)

F'_c, F'_q = Vesic cavity expansion factors (Fig. 16)

H = vertical height (or increment) of stone column treated ground over which settlements are calculated

H' = height of embankment in stability analysis (Fig. 46)

I_r = rigidity index used in Vesic cavity expansion theory (equation 13)

K_1 = proportionality constant for a normally consolidated clay between undrained shear strength and effective stress, $K_1 = c/\bar{\sigma}$

K_o = coefficient of at-rest earth pressure

K_p = coefficient of passive earth pressure

k_r = permeability of soil in radial direction (Fig. 45)

k_v = permeability of soil in vertical direction

k_s = permeability of smear zone in radial direction (Fig. 45)

L = length of stone column

M_d = driving moment in a stability analysis (equation 44)

M_r = resisting moment in a stability analysis (equation 44)

N = number of drainage surfaces at the top and bottom of the layer (N = 1 or 2); also normal force on lower failure surface in stone column undergoing local bearing failure (Appendix B)

$\tilde{N}_c$ = ultimate bearing capacity factor of stone column (equation 50)

n = stress concentration factor, σ_s/σ_c (Fig. 14)

η = reduction factor for local bearing failure of a stone column (Appendix B)

n^* = ratio of the unit cell radius to the radius of the drain (stone column radius less smear zone thickness), $n^* = r_e/r_w$ (Fig. 43)

n^*_{eq} = equivalent value of n^* for a drain without smear, $n^*_{eq} = r_e/r^*_w$ (Fig. 44)

P_H = ultimate lateral resistance of clay acting on critical wedge for a local bearing failure of stone column (Appendix B)

P_R = ratio of load carried per column in a group loaded by a rigid plate to the load carried by a single column loaded by a rigid plate having the same tributary area as one column in the group

q = mean isotropic stress, $q = (\sigma_1 + \sigma_2 + \sigma_3)/3$

q_{ult} = ultimate bearing capacity

$\tilde{q}_{ult}$ = ultimate bearing capacity of stone column

r_e = radius of the unit cell (Fig. 45)

r_s = radius of smear zone (Fig. 45)

r_w = radius of the drain usually taken as the radius of the stone column less the thickness of the smear zone (Fig. 45)

r_w^* = radius of equivalent drain without smear (Fig. 45)

S = settlement of unimproved ground

S* = smear factor used in radial consolidation theory, $S = k_r(s^*-1)/k_s$

S_g = settlement of a stone column group (Fig. 50)

S_1 = settlement of a single stone column (Fig. 50)

S_t = settlement occurring in an increment H of stone column treated ground

S_t' = primary consolidation settlement at time t

s = center to center spacing of stone columns (Fig. 13)

s* = ratio of the radius of smear zone to radius of the drain, $s^* = r_s/r_w$ (Fig. 44)

T = shear force on lower failure surface in stone column undergoing local bearing failure (Appendix B)

$\tilde{T}$ = assumed thickness of fictitious strip of soil used to obtain proper stress concentration in a computer stability analysis (Fig. 46)

T_r = time factor for radial drainage, $T_r = C_{v_r} t/D_e)^2$ (Fig. 43)

T_z = time factor for vertical drainage, $T_z = c_v t/(H/N)^2$ (Fig. 42)

U = average degree of consolidation considering both vertical and radial drainage, $U = 1 - (1-U_z)(1-U_r)$

U_z = average degree of consolidation in vertical direction (Fig. 42)

U_r = average degree of consolidation in radial (horizontal) direction (Fig. 43)

$\overline{W}_N$ = effective normal force exerted on upper failure wedge-local bearing failure (Appendix B)

W_s = effective weight of stone in failure wedge-local bearing failure of stone column (Appendix B)

$\bar{W}_v$ = effective vertical force exerted on the circular arc failure surface or the upper surface of the failure wedge for local bearing failure, (Appendix B)

w = width of equivalent, continuous stone strip used in a stability analysis $w = A_s/s$ (Fig. 46)

z = depth below ground surface

α = inclination of lower failure surface in a stone column undergoing a local bearing failure (Appendix B)

β = inclination of shear surface with respect to the horizontal

γ_{avg} = average unit weight of material within unit cell

γ_c = saturated (wet) unit weight of cohesive soil

$\bar{\gamma}_c$ = bouyant unit weight of cohesive soil

γ_f^c = weight of fictitious soil strip for use in computer stability analysis, $\gamma_f^c = (\mu_c - 1)\gamma_1 H/\tilde{T}$ (Fig. 46)

γ_f^s = weight of fictitious soil strip for use in computer stability analysis, $\gamma_f^s = (\mu_s - 1)\gamma_1 H/\tilde{T}$ (Fig. 46)

γ_s = saturated (wet) unit weight of stone column

$\bar{\gamma}_s$ = bouyant unit weight of stone in failure wedge-local bearing failure

γ_1 = unit weight of embankment in stability analysis (Fig. 46)

Δc_t = increase in undrained shear strength with time due to consolidation (equation 46)

ν = Poisson's ratio

ν_c = Poisson's ratio of soil

ν_s = Poisson's ratio of stone column

ψ = $\tan\alpha - \tan\beta$ (equation 56, Appendix B)

μ = reduction factor to apply to measured field vane shear strengths (Fig. 73)

μ_c = ratio of stress in cohesive soil to average stress, $\mu_c = \sigma_c/\sigma$, equation 8a

μ_s = ratio of stress in stone column to average stress, $\mu_s = \sigma_s/\sigma$, equation 8b

σ = average stress acting over the unit cell area due to the applied loading (Fig. 14); stress distribution should be considered with depth where important (Fig. 40)

σ_c = average stress acting over the soil in the unit cell (Fig. 14)

σ_i = average stress σ acting over the unit cell area in stone column improved ground at depth i (Fig. 40)

σ_s = average stress acting over the stone column (Fig. 14)

σ_1 = major principle stress

σ_3 = minor principle stress

$\bar{\sigma}$ = effective overburden stress

$\bar{\sigma}_o$ = initial effective stress in cohesive soil before stone column construction

σ_{va} = average of initial and final stress state applied to the coehsive soil; σ_{va} is used in equation 47 to calculate E from consolidation test results

τ_c = shear strength in coehsive soil on failure surface in a stability analysis

τ_s = shear strength in stone on failure surface in a stability analysis

ϕ_s = angle of internal friction of stone column

ϕ'_s = reduced angle of internal friction of stone to approximately consider local failure, $\tan\phi'_s \cong 2/3 \tan\phi_s$

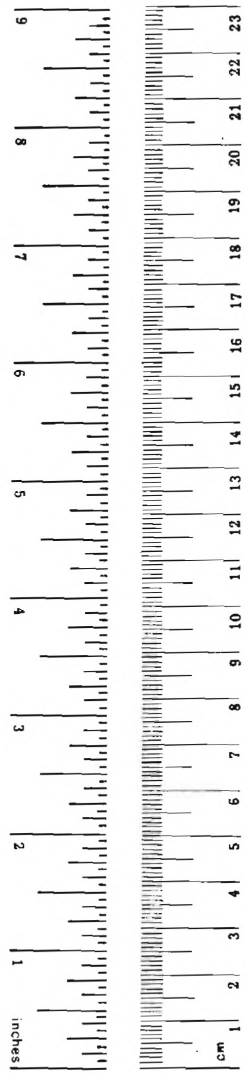
ϕ_c = angle of internal friction of soil

METRIC CONVERSION FACTORS

Approximate Conversions to Metric Measures

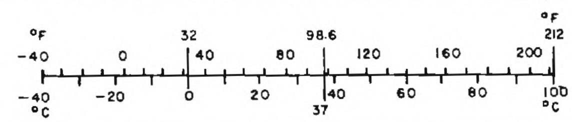
Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
in	inches	*2.5	centimeters	cm
ft	feet	30	centimeters	cm
yd	yards	0.9	meters	m
mi	miles	1.6	kilometers	km
		AREA		
in^2	square inches	6.5	square centimeters	cm^2
ft^2	square feet	0.09	square meters	m^2
yd^2	square yards	0.8	square meters	m^2
mi^2	square miles	2.6	square kilometers	km^2
	acres	0.4	hectares	ha
		MASS (weight)		
oz	ounces	28	grams	g
lb	pounds	0.45	kilograms	kg
	short tons (2000 lb)	0.9	tonnes	t
		VOLUME		
tsp	teaspoons	5	milliliters	ml
Tbsp	tablespoons	15	milliliters	ml
fl oz	fluid ounces	30	milliliters	ml
c	cups	0.24	liters	l
pt	pints	0.47	liters	l
qt	quarts	0.95	liters	l
gal	gallons	3.8	liters	l
ft^3	cubic feet	0.03	cubic meters	m^3
yd^3	cubic yards	0.76	cubic meters	m^3
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5/9 (after subtracting 32)	Celsius temperature	°C

*1 in ≡ 2.54 (exactly). For other exact conversions and more detailed tables, see NBS Misc. Publ. 286, Units of Weights and Measures, Price $2.25, SD Catalog No. C13.10:286.



Approximate Conversions from Metric Measures

Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
mm	millimeters	0.04	inches	in
cm	centimeters	0.4	inches	in
m	meters	3.3	feet	ft
m	meters	1.1	yards	yd
km	kilometers	0.6	miles	mi
		AREA		
cm^2	square centimeters	0.16	square inches	in^2
m^2	square meters	1.2	square yards	yd^2
km^2	square kilometers	0.4	square miles	mi^2
ha	hectares (10,000 m^2)	2.5	acres	
		MASS (weight)		
g	grams	0.035	ounces	oz
kg	kilograms	2.2	pounds	lb
t	tonnes (1000 kg)	1.1	short tons	
		VOLUME		
ml	milliliters	0.03	fluid ounces	fl oz
l	liters	2.1	pints	pt
l	liters	1.06	quarts	qt
l	liters	0.26	gallons	gal
m^3	cubic meters	35	cubic feet	ft^3
m^3	cubic meters	1.3	cubic yards	yd^3
		TEMPERATURE (exact)		
°C	Celsius temperature	9/5 (then add 32)	Fahrenheit temperature	°F



CHAPTER I

INTRODUCTION

Because of the ever increasing value of land, the development of marginal sites, once cost prohibitive, is now often economically feasible. The increased cost of conventional foundations and numerous environmental constraints greatly encourage the in-situ improvement of weak soil deposits. To economically develop marginal sites a number of new ground improvement techniques have been recently developed [1,3,5,6,7,67]. Some of these techniques are feasible for present use, but many require considerable additional research. Nevertheless, an important need now exists for proven techniques which can be used as environmentally acceptable and economically viable alternatives to conventional foundation support systems.

Construction of highway embankments using conventional design methods such as preloading, dredging, and soil displacement techniques can often no longer be used due to environmental restrictions and post-construction maintenance expenses [6]. Stone columns are one method of ground improvement having a proven record of experience. They are ideally suited for improving soft clays and silts and also for loose silty sands. Apparently, the concept was first applied in France in 1830 to improve a native soil [1]. Stone columns have been in somewhat limited use in the U.S. since 1972. However, this method has been used extensively in Europe for site improvement since the late 1950's.

PURPOSE OF STUDY

The purpose of this study is to synthesize the current state-of-the art of stone column construction and design. To compile available information on stone columns, fact finding trips were made within the United States and also to Canada, Europe and Asia. Small-scale model tests were conducted and theory developed to supplement existing knowledge concerning the behavior mechanisms and design of stone columns. Throughout the report, emphasis is placed on the practical aspects of stone column design, construction, inspection and testing.

A detailed discussion of the construction, utilization, and limitations of vibro-replacement and vibro-displacement stone columns is given in Chapter II. Failure mechanisms and analytical theories for predicting stone column performance are presented in Chapter III. Chapter IV contains a summary of subsurface investigation and laboratory testing techniques associated with stone columns. A set of guide specifications for the construction of stone columns is given in Chapter V together with detailed construction inspection guidelines. Selected case histories illustrating the use of stone columns are given in Chapter VI. Finally, Chapter VII synthesizes the practical results of the study as related to design. Specific recommendations are given in this chapter for the design of stone columns including its applications and limitations.

The report is written so that each chapter is essentially independent of the others. Therefore, in reading the report, chapters can be omitted as desired without losing continuity.

STONE COLUMN CONSTRUCTION

Stone column construction involves the partial replacement of unsuitable subsurface soils with a compacted vertical column of stone that usually completely penetrates the weak strata. When jetting water is used the process is named vibro-replacement (or the wet process). When used without jetting water in partially saturated soils, such as old rubble fill, the process is known as vibro-displacement (or the dry process). To date only the wet process has been used in the U.S., although both the wet and dry processes have been used in Canada and Europe. These techniques have been used since the late 1950's to construct columns of stone in marginal soils.

The stone is densified by the use of a vibrating probe originally developed in 1935 for the compaction of granular, noncohesive soils [13]. Although each specialty contractor identifies their vibrator by a different name, the term Vibroflot or Poker is frequently used to describe the probe. Rotation of eccentric weights within the body of the probe using either electric or hydraulic power causes lateral vibration at the tip of the probe. In the wet process the Vibroflot opens a hole by jetting using large quantities of water under high pressure. In the dry process, which may utilize air, the probe displaces the native soil laterally as it is advanced into the ground. In both methods the weight of follower tubes attached above the probe and the vibration of the probe aid in advancing the hole.

The probe typically varies in diameter from 12 to 18 in. (300-460 mm) depending on the individual contractors' equipment. Due to soil erosion and lateral compaction, the excavated hole is slightly larger than the probe. To construct the column, the hole is backfilled in 1 to 4 ft. (0.3-1.2 m) lifts with the probe usually being left in the hole. Stone is dumped from the ground surface and allowed to fall through the annular space provided between the probe and the sides of the enlarged hole. In soils which will not collapse, the probe is sometimes removed before adding the stone. Each lift is repenetrated several times with the vibrating probe to densify the stone and force it into the surrounding soil. The vibrating probe may also be momentarily left in a stationary position to densify the stone. Successive lifts are placed and densified until a column of stone has been formed up to the ground surface of the native soil.

MECHANISM OF PERFORMANCE

In stone column construction, usually 15 to 35 percent of the weak soil volume is replaced by stone. Design loads on stone columns typically vary from 20 to 50 tons. The presence of the column creates a composite material of lower overall compressibility and higher shear strength than the native soil alone. Confinement, and thus stiffness of the stone, is provided by the lateral stress within the weak soil. Upon application of vertical stress at the ground surface, the stone and weak soil move downward together resulting in an important concentration of stress within the stone column. The resulting stress concentration in the stone is primarily due to the column being stiffer than the soil.

An axial load applied at the top of a single stone column produces a large bulge to a depth of 2 to 3 diameters beneath the surface. This bulge, in turn, increases the lateral stress within the clay which provides additional confinement for the stone. An equilibrium state is eventually reached resulting in reduced vertical movement when compared to the unimproved soil. Stone column groups loaded over the entire area undergo less bulging than for a single stone column.

When an embankment is constructed over soft ground, lateral spreading of the ground occurs beneath the embankment which reduces the confinement of the stone column. At higher stress levels relative displacement (slip) may also occur between the stone column and surrounding soil. The occurrence of either lateral spreading or slip results in greater settlement of stone column improved ground than would otherwise occur.

STONE COLUMN USES

The stone column technique of ground treatment has proven successful in (1) improving slope stability of both embankments and natural slopes, (2) increasing bearing capacity, (3) reducing total and differential settlements, (4) reducing the liquefaction potential of sands and (5) increasing the time rate of settlement. Stone columns are used to support structures overlying both very soft to firm cohesive soils and also loose silty sands having greater than about 15 percent fines. At the present time, more stone column projects in the U.S. have been constructed in silty sands rather than cohesive soils; worldwide the reverse is true.

Previous Applications

Stone columns have been used successfully in the U.S. before 1982 on 21 projects including the following applications [68]:

1. Embankment Fill Support - for highways, interchanges and bridge approaches.

2. Miscellaneous Highway Facilities - hospitality station, box culvert.

3. Structures - seven-story concrete library, two-story medical building, warehouses, shipbuilding facility, sewage treatment plant, parking garage, miscellaneous office buildings.

4. Tanks - LGN storage tank, five million gallon water storage tank.

5. Miscellaneous - railroad and wharf structure.

In Europe stone columns have been used considerably more extensively than in either the U.S. or Canada. In England stone columns have been used to support about 40 bridge abutments. In France an approach fill and reinforced earth abutment have been constructed over a soft clay reinforced with stone columns. In general, however, stone columns have been used more extensively in Europe for the support of structures such as warehouses, tanks and buildings rather than embankments.

Sand compaction piles are similar in general concept to stone columns. The difference, however, is that sand compaction piles are constructed by vibrating a closed end pipe to the required depth. As the pipe is subsequently extracted from the ground, the hole is filled with sand. These piles offer an alternative to stone columns, particularly for embankment support. In Japan, they have been used extensively for the support of fills, embankments, tanks, and structures [24,66].

Potential Highway Uses

Important potential uses of stone columns for highway applications are as follows:

1. Embankments. The use of stone columns (or sand compaction piles) offers a practical alternative for the support of highway embankments where conventional embankments cannot be constructed due to stability considerations. Potential applications include moderate to high fills on soft soils and for fill, perhaps of Reinforced Earth, constructed on slopes where stability cannot otherwise be obtained. Stone columns were used at Hampton, Virginia [27] and also Clark Fork, Idaho [10], for the reasons just given although environmental factors were also an important consideration at Hampton, Virginia. Landslides are also an important potential application.

   A considerable amount of widening and reconstruction work will be done in future years. Some of this work will involve building additional lanes immediately adjacent to existing highways constructed on moderate to high fills over soft cohesive soils such as those found in marsh areas. For this application differential settlement between the old and new construction is an important problem in addition to embankment stability. Support of the new fill on stone columns offers a viable design alternative to conventional construction.

2. Bridge Approach Fills. Stone columns can be used to support bridge approach fills, to provide stability, and to reduce the costly maintenance problem at the joint between the fill and bridge. Stone columns have been used at Lake Okoboji, Iowa and Mobridge, South Dakota for a bridge approach and embankment, respectively. At Sioux City, Iowa, stone columns were used for an interchange [68]. Under favorable conditions stone column supported embankments can

be extended outward over wide, soft marsh areas and along rivers and lakes further than a conventional approach embankment. The potential therefore exists of reducing the length of costly bridge structures and by the use of stone columns.

3. Bridge Abutment and Foundation Support. Stone columns can be used to support bridge abutments at sites which are not capable of supporting abutments on conventional shallow foundations. At such sites an important additional application involves the use of a Reinforced Earth abutment supported on stone columns as was done at Rouen, France [63]. Of course, conventional reinforced concrete abutments can also be supported on stone columns as has sometimes been done in England. These abutments may or may not support the bridge superstructure.

   Another potentially cost effective alternative to pile foundations for favorable site conditions is to support on stone columns, single span bridges, their abutments and, if required, the approach fills. This technique minimizes the differential settlement between the bridge and approach fill.

   At grade separation sites underlain by marginal soils with respect to the use of shallow foundations, stone columns also offer a design alternative for the support of bridge pier foundations.

4. Liquefaction. In earthquake prone areas stone columns can be used to reduce the liquefaction potential of cohesionless soils supporting embankments, abutments and beneath shallow foundations. Stone columns can also be used to reduce the liquefaction potential of cohesionless soils surrounding existing or proposed pile foundations. Stone columns have been used, for example, at Santa Barbara, California [30,81] to reduce the liquefaction potential and also decrease foundation settlement. Stone columns have also been used at Kavala, Greece [126] to reduce liquefaction potential.

CONCLUSIONS

Stone columns are ideally suited for improving soft silts and clays and loose silty sands. Stone columns offer a valuable technique under suitable conditions for (1) increasing bearing capacity and slope stability, (2) reducing settlement, (3) increasing the time-rate of consolidation, and (4) reducing liquefaction potential. Applications of stone columns include the support of embankments, abutments, bridges and other type structures. Stone columns can also be used for stabilizing existing slopes. The use of stone columns to support Reinforced Earth structures results in a flexible type of construction which may be quite economical.

For each specific application, however, stone columns should be carefully compared with other design alternatives considering both the economics, advantages, and limitations of each method. Finally, stone column construction is part an art, requires careful field control and an experienced contractor.

CHAPTER II

PRESENT STATUS OF VIBRO–COMPACTED STONE COLUMN CONSTRUCTION

INTRODUCTION

This chapter summarizes present practices and equipment used to construct stone columns. The limitations of stone columns are also discussed. This review of stone column design and construction practices in the United States and Europe was developed from both literature and extensive interviews with engineers and specialty contractors in the U.S., Europe and Asia. The companies and individuals participating in the interviews are given in Appendix A of this report together with their addresses.

FEASIBILITY OF STONE COLUMN UTILIZATION

The construction technique for stone columns is well documented [15,29] and used extensively in Europe for economical stabilization of "soft soil" sites. Table 1 presents a summary of the opinions of selected contractors and engineers regarding the applicability of stone columns for various foundation treatments and site conditions as well as limitations and comments on the technique and the current technology. Some generally slight differences in opinions exist among the various individuals and organizations. A generalized summary of the factors affecting the feasibility of stabilizing soft ground with stone columns is as follows:

1. One of the best applications of stone columns is for stabilizing large area loads such as embankments, tank farms, and fills for overall stability and the control of total and differential settlements.

2. The design loading on the stone column should be relatively uniform and limited to between 10 and 50 tons per column.

3. The most improvement is likely to be obtained in compressible silts and clays occurring near the surface and ranging in shear strength from 300 to 1000 psf (15-50 kN/m^2). The greatest economic advantage is generally realized if the depth to the bearing strata is between about 20 and 30 ft. (6-10 m).

4. Special care must be taken when using stone columns in sensitive soils and in soils containing organics and peat lenses or layers. Because of the high compressibility of peat and organic soils, little lateral support may be developed and large vertical deflections of the columns may result. When the thickness of the organic layer is greater than 1 to 2 stone column diameters, vibro-

TABLE 1A. APPLICABILITY AND LIMITATIONS OF STONE COLUMN USAGE - PART 1[1,2].

Contractor/ Consulting Firm	Best Application of Technique	Problem with Technique	Limitations of Stone Column Usage	Comments on Existing Methods and Future Needs
Vibro-Constructed Stone Columns				
GKN Keller Ltd.	Fill, embankments, area stabilization, industrial sites; δ control; for 15-50 ft. lengths; low rise housing foundations; reduce liquefaction potential	Peat layers >3 ft. thick; Insufficient flushing water; Overstress; Misuse; Expect too much from system; Inadequate soil investigation	Careful with peat; Not applicable in refuse; Limit 40 - 50 ton/col. max.	Instrumentation of stress distribution & load transfer with time needed for large scale project; Use of scale model results with caution; Use for stability of fill & excavation
Cementation Piling and Foundations, Ltd.	Oil tanks, embankments, rigid mult-story str.; Structures/ projects not sensitive to δ	For silts and sensitive soils use wet technique and come in and out quickly	Limit: c = 400-800 psf (used in soils of c = 150 psf ; 5 - 15 ton/column (clay); 15 - 80 ton/column (sandy soil)	Full-scale testing best; More settlement readings needed; Settlements typically reduced by 1/2; Careful with FEM results
Karl Bauer Spezialtiefbau Gmbh.	Foundation stabilization, 1 - 2 story bldgs.; fill support, δ control; 20 ft. length most economical	Heavy (3 - 5 story) bldgs., irregular loads, bridge fnds.; Soil report errors, old equipment, new crew; Peat lenses	Probs. in sensitive silts and soft organics; Very soft soil; Limit 10 - 40 ton/col. (avg. 15 - 25 ton/ col.)	Field data needed; careful with abutments; E improved by 1.5 to 4; δ reduced 20 - 30%; FEM may be useful; Study single + group effect with time.
Vibroflotation Foundation Co. & Vibroflotation (U.K.) Ltd.	Embankment, abutments, area stabilization, fdns., slope stability; Reduce effect of soil variability w/compacted mat; Limit δ	Shear strength < 150 psf; Large diameter S.C.; Overload	Organic layer < 3-10 ft. thick; c = 200 to 400 psf; Length < 15 - 40 ton/col. max.	Monitor more full-scale projects; Better analysis needed; FEM potentially powerful tool; Earthquake resistance a potential use
Franki Pile Company	Reduce total δ and differential δ; Stock-piles, warehouses including floor slab, footings, oil tanks, Slope stability	Stability problems in cohesionless and soft clays when GWT high; Good soil description, CPT and pressuremeter tests.	Problems - stability during construction and bulging; Need preliminary assessment of bulging load; Use rammed cols. if stability is problem	Field instrumentation to better understand load transfer and deformations

Notes: 1. Table 1 is continued on the following page.

2. Notation Used: GWT = groundwater table; S.C. = stone column; FEM = finite element method; δ = settlement; w/ = with; bldg. = building; engr. = engineer; cont. = contractor; c = shear strength of soil.

3. Unit Conversions: 1 ft. = 0.305 m; 1 psf = 47.9 N/m^2.

TABLE 1B. APPLICABILITY AND LIMITATIONS OF STONE COLUMN USAGE - PART 2[1].

Contractor/ Consulting Firm	Best Application of Technique	Problem with Technique	Limitations of Stone Column Usage	Comments on Existing Methods and Future Needs
Landesgewer-beanstalt - Bayern (LGA)	Oil tanks, embankments, warehouses, single story uniformly loaded bldgs.; δ reduction; Good for weak layer > 13 ft. and high GWT	Peat; cooperation w/ engr./cont.; Soil report deficient; Floating columns; Excessive loads; Change in construction plans and spacing; Alignment of S.C.; Small diameter S.C.; Soil not suitable	c from 300-1000 psf	Analysis needs improvement; Not for abutments; Improvement by factor of 2
Institut fur Grundban Bodenmedanik	Rigid loading (raft); tank farms; Stabilize top zone (3 ft.) w/mat; Use for abutment if S.C. @ 30°; δ and differential δ control	Assume total & differential δ; Floating S.C. columns; Weakens sensitive soils	c > 300-500 psf c < 1-2 ksf) Structure of peat important	Analysis needs improvement; 12 in. diameter triaxial model appropriate; Cost effectiveness questioned
Thorburn & Partners	Strip footings, houses, factories; Reduce effect of soil variability; Act as drain; Use compacted mat to stiffen col. & surface; Design structure to handle differential δ	Contamination of stone; Unconfined peat layers; Misuse	c limit 400-1000 psf; Organic silts, clays and peats at surface are problem; Refuse is problem	Not recommended for stability or embankments; Abutments are possible; Full-scale FEM, model tests needed
		Rammed Stone Columns		
Datye, et al. Rammed Stone Column	Stability; Preload for δ; Industrial bldg; Bridge approach transition; floating S.C. sometimes used for stability	Method of const./instal. critical to performance; Gap grade sand/stone to prevent segregation	Used for c $\gtrsim$ 100 psf	Better performance reported than for Vibro S.C.; Rammed S.C. used on only a few projects
Franki Pile Company - Rammed Stone Column[2]	Reduce total & differential δ; Stockpiles, warehouses including floor slab, bldg. footings, oil tanks; Slope stability; decrease lateral soil displacement	Beneath GWT soft and low permeable soil can penetrate stone at high load - use finer gradation or sand.	20 to 60 tons/col.; Bulging is limiting condition; Sand gives more δ than stone, but forms a filter	Franki uses cased hole compared to open hole of vibro technique; Capacity similar to Vibro of Franki hole and casing dia. similar; Franki capacity greater if rammed S.C. dia. 4 in. greater than dia. of casing.

Notes: 1. Notation Used: GWT = groundwater table; S.C. = stone column; FEM = finite element method; δ = settlement; w/ = with; bldg. = building; engr. = engineer; cont. = contractor; c = shear strength of soil.

2. Rammed stone columns are constructed by Franki Pile Co. primarily in Belgium. They use a vibrator to construct stone columns in other parts of Europe, South Africa and Australia.

3. Unit Conversions: 1 ft. = 0.305 m; 1 in. = 25.4 mm; 1 psf = 47.9 N/m^2.

replacement should not be used. When thick peak deposits are encountered two vibrators are sometimes fastened together to keep the ratio of layer thickness to column diameter within allowable bounds.

When used under the ideal conditions previously described, stone columns for certain conditions may be more economical than conventional alternatives such as complete replacement, and bored or driven piles. Ground improved with stone columns is believed to give settlements typically varying from 30 to 50 percent of the unimproved ground response. As discussed in Chapter III and VII, however, actual reductions in settlement are often somewhat less than generally believed. An important secondary benefit of stone columns at favorable sites is that the time-rate of settlement is significantly decreased compared to unimproved ground.

The length of stone columns used in Europe tend to be generally between 13 and 33 ft. (4-10 m). Complete removal and replacement, which is an alternative to stone columns, is usually practical and economical for depths less than about 20 ft. (6 m). Stone column depths greater than about 30 ft. (10 m) are usually not economically competitive with conventional deep foundations. Furthermore, construction of very deep stone columns is considered by many to pose serious construction problems including stabilization of the hole and insuring that uncontaminated stone gets to the bottom and is properly densified. However, both European and American contractors have experience in the design and construction of stone columns as long as 70 ft. (21 m) with few problems being reported. Nevertheless, considerable caution should be exercised in constructing long stone columns.

Stone columns have been used in soils having minimum (not average) undrained shear strengths as low as about 150 psf (7 kN/m^2). The contractors agree that the fabric or structure of peat-like soils influence the lower allowable limit. A practical upper limit, due to the development of excessive resistance to penetration of the vibrator and economic considerations, is in the range of an undrained strength of 1000 to 2000 psf (50-100 kN/m^2). Soils with greater shear strengths may, in fact, be strong enough to withstand the loads without ground improvement. If ground stabilization is required in these stiff soils or through stiff lenses, the hole is frequently prebored, which is often the case in landslide projects.

CONSTRUCTION OF STONE COLUMNS

The improvement of a soft soil with stone or sand columns can be accomplished using various excavation, replacement and compaction techniques. The principal construction methods, some of the firms that use these techniques and typical site conditions where the techniques are used are as follows:

Vibro-Replacement (wet): In the vibro-replacement (wet) method, a hole is formed in the ground by jetting a probe down to the desired depth. The uncased hole is flushed out and then stone is added in 12 to 48 in. (0.3-1.2 m) increments and densified by means of an electrically or hydraulically actuated vibrator located near the bottom of the probe. Stone columns are presently constructed in this way by GKN Keller, Ltd. (Worldwide), Karl Bauer Spezialtiefbau GmbH (Europe, Middle East), Vibroflotation Foundation Company (Worldwide), and Cementation and Cementation Franki (Worldwide). The wet process is generally used where borehole stability is questionable. Therefore, it is suited for sites underlain by very soft to firm soils and a high ground water table.

Vibro-Displacement: The vibro-displacement method is a dry process sometimes referred to as vibro-replacement (dry). The main difference between vibro-displacement and vibro-replacement is the absence of jetting water during initial formation of the hole in the vibro-displacement method. Most contractors can use either the wet or dry process. To be able to use the vibro-displacement method the vibrated hole must be able to stand open upon extraction of the probe. Therefore, for vibro-displacement to be possible soils must exhibit undrained shear strengths in excess of about 850 to 1250 psf (40-60 kN/m^2), with a relatively low ground water table being present at the site.

In the past several years GKN Keller Ltd. and Karl Bauer Spezialtiefbau GmbH. have developed the capability to stabilize sites underlain by soft soils and high ground water using the dry process. Stabilization is made possible by using a new "bottom feed" type vibrator. Eccentric tubes adjacent to the probe allow delivery of stone, sand or concrete to the bottom of the excavated hole without extracting the vibrator. Using this method the vibrator serves as a casing which prevents collapse of the hole.

Rammed Stone Columns: Rammed stone columns are constructed by either driving an open or closed end pipe in the ground or boring a hole. A mixture of sand and stone is placed in the hole in increments, and rammed in using a heavy, falling weight [52-55,73,107,108]. Cementation Franki (formerly Franki Pile Co.) constructs rammed stone and also rammed sand columns primarily in Belgium. The consulting firm of Dubon Project Engineering PVT, Ltd., headed by K. R. Datye, has developed several techniques for the construction of rammed stone columns in India. Since a casing is initially placed into the subsurface soils, potential hole collapse is eliminated. Therefore, the technique has application in most soils treatable by the vibro-techniques. Disturbance and subsequent remolding of sensitive soils by the ramming operation, however, may limit its utility in these soils. A more detailed consideration of Franki rammed stone and sand columns is given in Appendix F.

Sand Compaction Piles: Sand compaction piles and several modifications to this technique are used extensively in Japan [24,66] and to a lesser extent in Taiwan. Sand compaction piles are constructed by driving a steel casing down to the desired elevation using a heavy, vertical vibratory hammer located at the top of the pile. As the pile is being driven the casing is filled with sand. The casing is then repeatedly extracted and partially redriven using the vibratory hammer. By the time the sand compaction pile has been completed the casing has been completely removed from the ground. Several variations of sand compaction pile construction procedures are used in Japan including placing a vibrator at the tip of the casing. The concept of initial hole formation is similar to the Franki system and thus subject to its limitations.

Sand compaction piles are used for stabilizing soft clays in the presence of high ground water. The Japanese, by varying equipment size and compaction energy, have developed three related systems which are selected based on the anticipated use, site conditions and loading [66].

In this chapter the installation procedures, equipment and special considerations are presented for the vibro-replacement and vibro-displacement methods of stone column construction. These are the two methods of constructing stone columns generally used in the western world at the present time. A general summary of the vibro method of construction is given in Tables 2 through 4. Design and construction of rammed stone columns and sand compaction piles as performed in Japan and Taiwan are described elsewhere [52-55, 66].

Vibrator

Stone columns are generally constructed using either an electric or hydraulically actuated, cylindrical shaped vibrating probe such as the one illustrated in Fig. 1 and 2. The vibrator, originally developed by Steuerman [13], essentially consists of a hydraulic or electric motor mounted within a cylindrical casing approximately 14 to 18 in. (360-460 mm) in diameter and 7 to 15 ft. (2.0-4.5 m) in length. The motor powers a set of rotating eccentric weights which provide the lateral vibration and compaction force. Depending on the specific unit selected, the lateral force varies from approximately 12 to 28 tons. Heavy wear plates are added to the sides of the vibrator protecting it from excessive wear during raising and lowering from the ground. Fins located on the sides of the vibrator prevent rotation. A small diameter vibration isolator is placed between the vibrator unit and the follower tubes. The heavy follower tubes serve the dual purpose of (1) providing the necessary vertical downward thrust to advance the probe and (2) providing an overall minimum length of about 33 ft. (10 m). Although the overall length can easily be increased by adding additional follower tubes, a 33 ft. (10 m) length is adequate for most applications.

The vibrator is suspended from the boom of a crane; a 33 ft. (10 m) probe can be easily handled using a 40 ton crane with a 40 ft. (12 m) boom. Penetration of the probe is accomplished by vibration, water jetting, and dead weight. New vibro units used by Keller and Bauer provide additional downward thrust by using hydraulic jacks attached to the boom and the probe. Such 'pull-down' units, illustrated in Fig. 3, are self-contained and have been used predominantly in Germany. The pull-down rig provides a good rate of installation, is of compact size, has bottom-feed capability, is self-contained, and does not require use of a crane.

At the present time the optimum amplitude and frequency of vibration for construction of stone columns has not been established. GKN Keller

TABLE 2. CHARACTERISTICS OF PROBES USED TO FORM STONE COLUMNS BY THE VIBRO METHOD.

Contractor	Installation Method	Weight(1) (tons)	Length (ft.)	Dia. (in.)	H.P.	Freq. (rpm)	Lateral(2) Force (tons)	Free Amp. (in.)
GKN Keller	Vibro; very seldom preauger; wet, dry bottom-feed	2.4	<15	12	46-66	3,000	12 - 15 (T Model)	0.28
		2.4		12	66	3,000	15 - 17 (Mohno)	0.28
		2.4		16	105	3,000	18 (A Model)	0.55
		2.4		12-22	160	1,800	28 (S Model)	0.63
Vibro-flotation	Vibro; preauger hard to penetrate stiff soil; No water in loess or clay shale	2	7.0	16	100	1,800	20	0.43
		2	6.11	15	30	1,800	10	0.30
		2	7.0	16	100	1,800	28	0.59
Cementation	Vibro; may preauger stiff crust	3-4	—	15.7 wet; 11.8 dry	Hydraulic	1,800	—	—
Bauer	Vibro; prebore only stiff lenses and surface crust	4; pull down rig also	10.5	12.8	Hydraulic	1,800-4,000	18(3)	Varies
Franki	Vibro	1.3	6.2	14.6	Hydraulic; 111 @ 2500 rpm	1,000-2,500	34 @ 2500 rpm	Varies

Notes: 1. Weight of vibrator section
2. Centrifugal Lateral Force developed by machine at operating speed.
3. At usual operating speed of about 3,000 rpm.

TABLE 3. INSTALLATION CHARACTERISTICS OF STONE COLUMNS FORMED BY THE VIBRO METHOD.

Contractor	Column Diam. (ft.)(1)	Column Install Rate	Jetting	Adj. Hole Collapse	Stone Backfill	Amps.
GKN Keller	2-3.5 avg.	30-60 ft/hr. (avg.)	Generally water; some problems with air	May cause prob. if too close; s>4 ft. (air) s>5 ft. (water)	3/8"-$1\frac{1}{2}$ in. generally; Softer matl's use 4 in. max.	70-100 amps; controls γ; amps used varies from site to site; do trial column to get value
Vibro-flotation	<4 typ.	40 ft/hr. (avg.) (soft soil 300-400'/ 8 hr.)	Water @ 100 psi; Cool electric vibrator	Some prob. in pre-augered hole; 6 ft. typ.	3/4 - 3 in. angular; some consultants prefer rounded	80 amps typ.
Cementation	~3	40-65 ft./ hr. (production)	Prefer dry method; Use wet if in doubt	Vary drill pattern; drill center last	3/4 - 2 in.; weathered, rounded, no laminate, hard; 3-4 in. if dry	Control const. time & quantity of stone; amps not as important
Bauer Germany	2.3 avg.	100 ft./ hr.	Use water; Air not as efficient	No prob. if s>4 ft.	Clean, strong broken material; 0.6(1.2)-2.75 in.; Stone filters soil	Use amps or hydraulic pressure; Permanent record costs extra
Franki	2.5-5	1 probe per hr.	Use Water	Spacing/ hole diam. ratio ≥ 2	0.2-1.25 or 2.5 in.; round stone	Control time and quantity of stone; Hydraulic pressure not critical

Notes: 1. The completed stone column diameter varies with the strength of the soil, equipment used, and method of construction (refer to Table 13).

2. Unit Conversions: 1 ft. = 0.305 m; 1 in. = 25.4 mm; 1 psi = 6.89 kN/m^2.

TABLE 4. CONSTRUCTION CONSIDERATIONS FOR VIBRO-CONSTRUCTED STONE COLUMNS.

Contractor	Special Considerations	Procedure	Flushing	Backfill Condition	Contamination	Stone @ Bottom	Problem Conditions
GKN Keller	1-3 in. Stone (dry); some consultants specify 50% split faces; Bottom-feed - use rounded stone	Vibrate hole, retract, dump stone (2 ft. lifts); vibro-w/water; Get good base by 2-3 repenetrations	Keep water running to wash fines & for stability; get stone to bottom; 3 flushes per hole	Min. fines; Front end loader can be used	Don't scrape ground (flushing hole will remove fines)	Construct slowly; keep flushing; Drive cone sometimes used(2)	NOT PERMITTED: (1) Peat > 3 ft. (2) Refuse in Fill
Vibro-flotation	Sand backfill would slow construction	Repenetrate to w/in 1-2 ft. of past level	Keep water flowing	Fines will wash out if water kept flowing	—	Follow procedure	NOT PERMITTED: Organics 3-7 ft. thick
Cementa-tion	Column top-carrot shaped, usually; Use compacted mat; Dig out and replace soil	Construct column in 1-2 ft. lifts	High water flow important	Be careful of G_s of stone	Don't scoop dirt up with stone	Drive cone sometimes used;(2)	Silts may liquefy - construct quickly in silt
Bauer Germany	Keep experienced engineer on site; Dry Method preferred (cleaner); wet method takes heavier load	Penetrate/flush 2-3 times; strong flushing required in peat	Keep water flowing	Not worried; use 1-3 ft. lift thickness	Not worried	More confidence with wet method	NOT PERMITTED: (1) soft organics; (2) decomposable material; construct fast in silt
Franki	When hole stability problem, rammed S.C. preferred	~ 1.6 ft. thick lift	Keep water over-flowing	—	—	Enlarge base	Rammed S.C. preferred from installation viewpoint in refuse, organics and peat

Notes: 1. Unit Conversions: 1 ft. = 0.305 m; 1 in. = 25.4 mm.

2. If problems are suspected, a 1 in. dia. drive cone is sometimes driven (or attempted to be driven) through the stone column; erratic results are usually obtained; frequently the cone does not reach the tip.

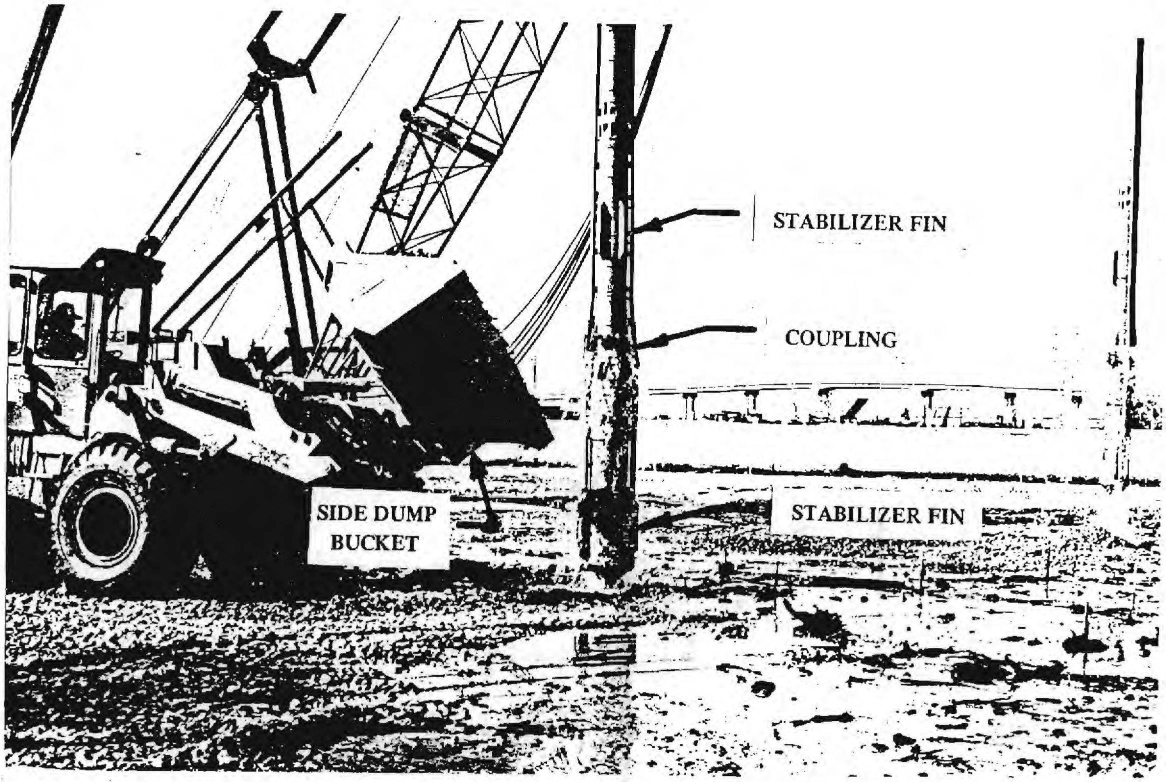


FIGURE 1. TYPICAL VIBRATOR AND END DUMP BUCKET USED BY VIBROFLOTATION FOUNDATION COMPANY - JOURDAN ROAD TERMINAL.

FIGURE 2. VIBRATOR USED BY FRANKI FOUNDATION COMPANY (Courtesy of Franki - Belgium).

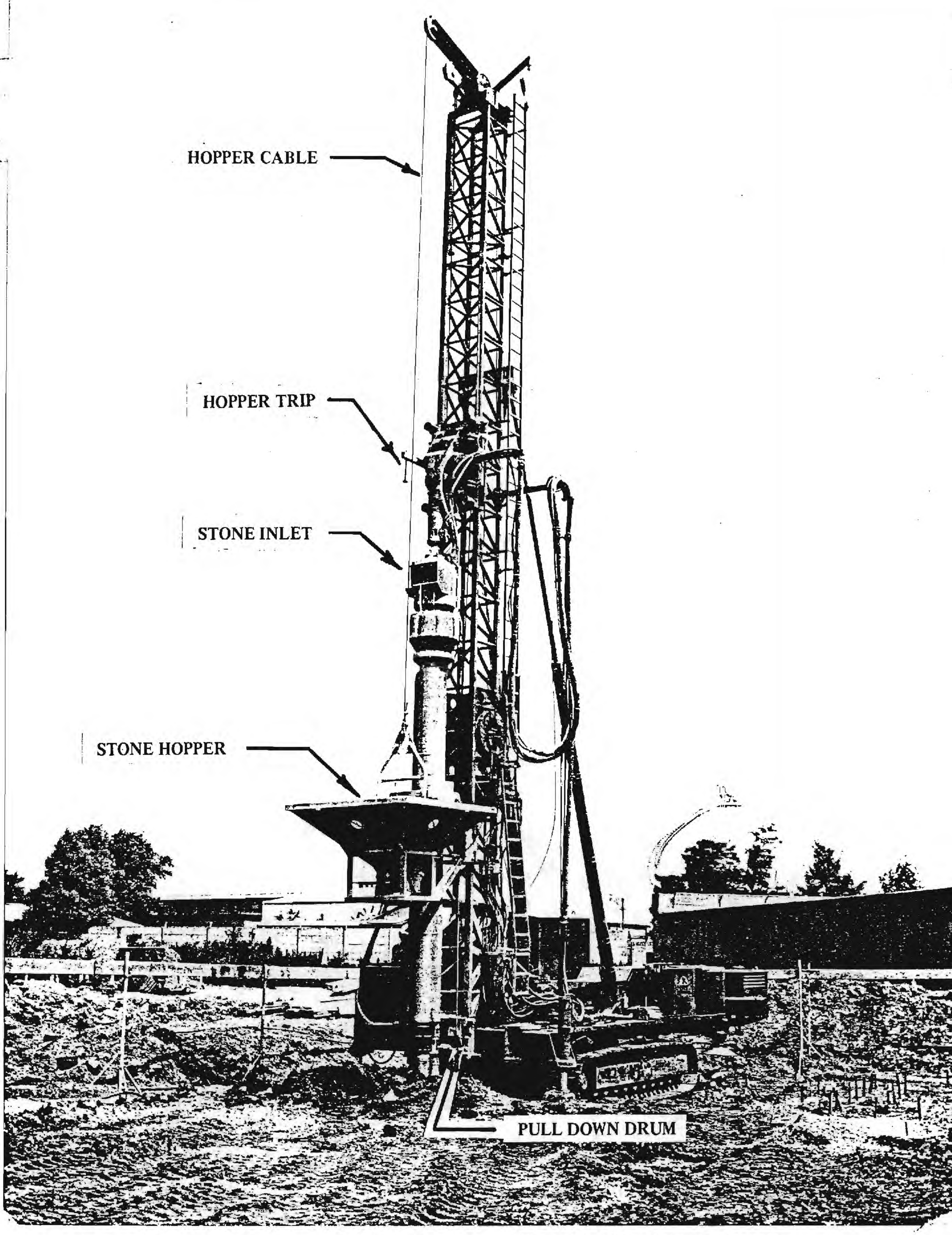


FIGURE 3. VIBROCAT PULL-DOWN RIG FOR STONE COLUMN CONSTRUCTION (Courtesy of GKN Keller, Inc.).

believes that high frequency and low amplitude is best for penetration of the soft soil and compaction; their units operate at 3000 rpm and 0.3 to 0.4 in. (7-10 mm) amplitude. On the other hand, Vibroflotation Co. and Cementation use a lower frequency (1800 rpm), higher amplitude unit (0.4 to 0.6 in.; 11-15 mm) because of the reportedly higher compaction efficiency. Bauer, who use a hydraulic unit, can continuously vary the vibration frequency. Typically a frequency of 2800 to 3200 rpm is used for both penetration and compaction; their experience indicates that the higher frequency units are also better for compaction. Frequencies have been used by them as high as 4000 rpm. However, for long machine life the frequency is usually limited to about 3200 rpm since excessive wear occurs on the motor bearings at high speeds.

The principal advantages of a hydraulic motor compared to an electric motor appears to be the ability to vary vibration frequency and safety considerations. Vibroflotation, Ltd. is currently developing a variable frequency electric motor. Unfortunately, a direct comparison is not available of the penetration and compaction efficiency and the resulting stone column strength obtained using various type, size and frequency vibrators. Over the years, however, each contractor has developed considerable experience with their machines, and have optimized construction procedures for their equipment and varying soil conditions.

Wet Installation Method

The details of construction using the vibro-replacement (wet) and vibro-displacement (dry) technique have been well documented by Thorburn [18], DiMaggio [9], Greenwood [15] and others. To date vibro-displacement (dry) stone columns have not been constructed in the U.S.; they have been used on two projects in Nova Scotia, Canada and numerous projects in Europe. The vibro-replacement (wet) method must be used at sites consisting of very soft soils unable to stand in an unsupported hole, and when high ground water conditions exist. Water jets at the bottom and along the sides of the unit facilitate both penetration of the vibrator and flushing loose soil from the hole. The flowing water also is important in stabilizing the hole and washing soil from the sides. Contractors usually prefer, where hole stability is suspect using the wet technique because the hole is supported during construction reducing the chance of a collapse. Also, the water used during the jetting operation cools the motor, which is important for electric powered units.

The principal disadvantage of the wet technique involves the large quantity of water which is required and which must later be disposed of without causing pollution. After being used for stone column construction, the water contains a significant quantity of suspended silt and clay. A large quantity of water should always be used in stone column construction to prevent collapse of the hole or contamination of the column; scarcity or high cost of water does not alter this requirement. Environmental regulations and low-lying or urban site conditions may restrict the drainage and disposal of the excess water-soil suspension. Unless properly handled by constructing sediment ponds, ditches and other drainage structures, pollution may occur. Further, standing pools of water may disrupt work and slow production.

Dry Installation Method

The vibro-displacement (dry) process is much cleaner than the previously discussed wet technique since it does not use jetting and flushing water. Although under certain conditions the dry process may be less expensive than the wet technique, actual site conditions must be carefully evaluated to insure that hole collapse is not a problem. The dry method is frequently used to carry stone columns through weak fills in developed areas because of the problems associated with the acquisition, retention, and disposal of significant amounts of water.

The dry technique is suited for partially saturated soils which can stand unsupported, especially those which will densify as a result of lateral vibration. Air is sometimes used as a jetting medium to facilitate extraction of the probe which occasionally adheres to the hole walls. Under suitable site conditions, contractors prefer the vibro-displacement (dry) process over vibro-replacement (wet). Owners also often prefer the absence of ponded, silt-laden water on site and may, in fact, realize an economic savings. However, when hole stability becomes uncertain water must be used. The question of economics must be considered on an individual basis. In general contractors have greater confidence in the consistency and integrity of stone columns formed using the wet process compared to the dry process. Also, the wet-formed columns are generally larger than their dry-formed counterparts. Therefore, the design load per column proposed by contractors for columns constructed using the wet process may be greater than for columns constructed using the dry process.

As additional experience is gained using the pull-down, bottom feed units (used thus far in Germany), the reservations concerning vibro-displacement construction may be relaxed. Although columns are formed dry, the probe remains in the hole at all time. Thus, the problem of collapse is eliminated, and the range of treatable soils is expanded to include soft silts and clays and high ground water conditions. The operation of the pull-down, bottom feed rig differs from the conventional probe as follows (Fig.3):

1. The pull-down unit is attached to the boom of a tractor mounted rig rather than hanging suspended from a crane. Hydraulic rams assist probe advancement into the subsurface soils, whereas heavy follower tubes assist the penetration of the crane-supported vibrator.

2. The maximum length of completed column constructed with the pull-down rig is somewhat fixed due to its attachment to the crawler boom. In the conventional system, the heavy follower tubes are also used to determine the length of the completed column.

3. Removal of the probe from the excavated borehole to facilitate the placement of stone is not necessary with the newer rig. Eccentric tubes mounted beside the probe permit stone to be added from a surface hopper and taken directly to the bottom of the hole. Optional use of compressed air atop the column of stone

which is contained within these tubes assists in placement of the stone and minimizes clogging within the tubes.

4. By injecting cement through the tube and into the voids within the stone, a rigid column may be formed. Concrete columns will be discussed in a subsequence section of this chapter and in Chapter VI.

The procedure of continuous repenetration of the stone with the vibrator remains unchanged. Thus utility of the pull-down, bottom feed system may represent an economic advantage compared to the conventional vibro-displacement system, particularly on smaller job sites having stone columns less than about 30 ft. (10 m) in length. Pushing the tube into the soil causes a continuous bearing failure at the tip which is closed end. The successive shear failures result in complete remolding of the soil around the probe, and also drags the soil downward immediately adjacent to the probe. The combined effects of this construction sequence is called smear. Smear due to pushing a closed end pipe results in an important reduction in the horizontal permeability [99] of the soil surrounding the probe and hence, ultimately around the stone column.

Stone Column Construction

The stone column is advanced to the required depth using either the wet or dry process. For foundation support, the base of the stone column should be carried down to a firm bearing strata rather than "floating" the column in soft soil. Contractors have more confidence in a column founded on a firm bearing material and feel that the bearing stratum foundation minimizes the potential for deep-seated settlements beneath the stone columns due to transfer of stress to the base of the column. For stability applications such as landslides, this requirement can be relaxed if caution is exercised. As discussed previously, it may be necessary to preauger stiff clays and silts which cannot be economically penetrated by the probe. Preaugering, however, is expensive since a drilling rig is required and hence is not a common practice.

After forming the hole to the required depth using the wet process, it is flushed out several times by raising and dropping the probe in the hole. Flushing the hole removes the silt and may slightly increase the diameter of the hole. Usually 2 to 3 flushings are adequate. In soils with organics or peat, however, proper flushing may require more surge cycles. These deleterious materials should, however, be flushed from the hole before proceeding on with construction.

After flushing, some contractors may occasionally remove the probe to facilitate stone placement, although most prefer to leave the probe in the ground at all times with the jets operating. Surging of the probe at this stage helps clean the stone and assists rapid placement of the stone. If hole stability is questioned the probe is always left in the ground with the water jets engaged. Specifications usually require the probe to remain in the hole at all times during construction.

Gradation of the stone used varies greatly depending upon the available sources of aggregate, subsurface conditions and the contractor. A range of successfully used gradation is given in Chapter V in the guide specifications. In general a coarse, open-graded stone is used, varying from about 0.5 to 3.0 in. (12-75 mm) in size. Crushed stone is preferred although natural gravel is also used. In Europe, brick rubble or concrete debris is frequently used, particularly in developed urban areas. A small amount of fines in the vibro-replacement stone presents no problems since it is flushed to the surface by the upward flowing water. For the dry method, a large stone up to 4.0 in. (100 mm) in size may be used to help insure it reaches the bottom. The uncertainty of the stone reaching the bottom of the hole highlights another potential problem with the dry construction procedure. The contractors can modify the construction procedure to accommodate well-graded as well as single sized gradations. Stone specifications for the bottom feed units include round to angular sand or gravel up to about 1.5 in. (40 mm) in diameter.

An important factor in successfully constructing stone columns is to keep water flowing from the jets at all times. This aids in stabilizing the hole and in washing soil (fines) from the hole to prevent it from settling out within the stone column. Sand cannot be used in columns constructed using the vibro technique because the large quantity of upward flowing water makes it difficult or impossible to get the light sand particles to the bottom of the hole.

The stone column is constructed in approximately 2 to 4 ft (0.6-1.2 m) lifts). The proper amount of stone is placed down the hole usually using an end dump bucket mounted on a front end loader. The previously placed stone is penetrated by the probe (which should have been left in the hole with jets running) several times to achieve good densification. As the probe densifies the stone, the power used by the vibrator motor generally increases.

Power consumption is commonly used as a guide to help insure proper densification of the stone. An ammeter and automatic recorder is frequently used to monitor and record power consumption during stone column construction if a permanent record is specified. Monitoring power consumption, however, does not alleviate the need for carefully inspecting the entire construction sequence. Indeed, some engineers feel a high power consumption simply insures good contact is achieved between the stone and probe. Good penetration of the probe as each lift is compacted should be considered equally important to a build-up of power consumption as this penetration is the mechanism for driving the compacted stone into the adjacent soft soil thus increasing the column diameter.

To construct a satisfactory stone column, a strong base of stone must be initially formed. Extra time should therefore be spent when stone is first added to the hole to fully penetrate the stone and create a large, well-compacted stone base upon which to build the remaining part of the column. Subsequent lifts are constructed by the addition of stone, and repeated penetration and retraction of the probe until the stone column is completed. The top section of the column is not subjected to excessive repenetration as the column near the top is generally larger due to the relatively low in-situ lateral soil resistance and soil erosion.

Because of lateral displacement of the stone during vibration, the completed diameter of the hole is always greater than its initial diameter. Typical hole diameters vary from about 2.5 to 4.0 ft. (0.8-1.2 m) depending upon the type soil, its undrained shear strength, stone size, characteristics of the vibrating probe, and the construction method. The diameter of the finished column is usually estimated using the stone take assuming a compacted density. Measurements that should be made to obtain a reasonable estimate of the diameter of the compacted stone column are summarized in the model specifications given in Chapter V. Occasionally test pits are dug in the soil adjacent to the stone column to determine its diameter and verify its integrity. These test pits often reveal a "carrot-shaped" column profile, with a bulge concentrated near the top of the column.

Subsequent stone columns are constructed by removal of the probe from the completed column and relocation of the crane to a predetermined adjacent location. The construction procedure is then repeated. Typically stone column spacing is approximately 6 to 9 ft. (1.8-2.7 m) although smaller spacing is possible. A minimum spacing of about 5 ft. (1.5 m) is imposed because of potential construction problems. As the spacing of the stone columns decrease, the amount of replaced soil rapidly increases. At close column spacing, the residual lateral forces surrounding the completed column may cause difficulty in maintaining the adjacent hole open during construction. These residual stresses, however, help provide lateral support for the constructed column. If a close spacing is used, a staggered construction sequence should be developed whereby alternate columns or groups are initially formed followed by the construction of the columns in between. The construction rate for stone columns depends upon the same factors that influence the completed diameter. In addition, the construction of stone columns can be greatly hindered by the presence of obstructions such as buried trees, boulders, hard lenses, and miscellaneous materials such as encountered in old fills. Average reported construction rates are 3 to 6 ft/min. (1-2 m/min) for excavation and 1.5 to 3 ft/min. (0.5-1 m/min.) for backfill and compaction.

RIGID STONE COLUMNS

In Europe for some applications cement has been added for about 10 years to the compacted stone column, thus forming a rigid column of concrete. GKN Keller and Karl Bauer Spezialtiefbau currently construct this type column with apparent success. The cost of rigid columns in the U.S. would be about $15 to $20/ft. ($50-$66/m) which is similar to conventional stone columns in price. The added cost of cement used in rigid columns is, approximately, offset by the faster construction time compared to conventional columns.

A brief discussion of the more important aspects of this technique is summarized as follows:

1. A rigid column is less dependent on lateral support supplied by the subsurface soils. Therefore, they can be used in very soft soils and are capable of carrying more load at smaller deformations

than their uncemented counterparts.

2. The technique can be applied to form a continuous rigid column or can be used to stiffen the stone column in weak zones where high lateral deformations are anticipated. Cement can therefore be applied to the stone through a weak layer with the remaining portion of the column consisting of uncemented stone. Load would thus be transmitted through the weak layer by the rigid column to the underlying stone column.

3. The load-deformation response of a rigid stone column is similar to a conventional pile. The ultimate load capacity can be more clearly defined than for a conventional stone column.

4. Construction of the rigid column generally follows a vibro-displacement (dry) process. A bottom feed unit capable of supplying cement or grout is well-suited for this process.

The mechanisms of performance of rigid stone columns are similar to conventional piles or piers. Therefore precast concrete piles, auger cast piles, timber piles and drilled piers in many applications such as foundation support would be direct competitors of rigid stone columns. Rigid stone columns appear to be best suited for (1) strengthening the stone column in locally weak zones and perhaps (2) for improving slope stability.

SPECIAL CONSIDERATIONS

Stone columns constructed using vibro techniques have been used extensively in Europe for about 20 years. During this time the contractors, as well as engineers using the technique, have developed rules-of-thumb as well as basic philosophy regarding their use and construction. Conventional stone columns are not recommended at sites which contain extensive refuse or decomposable organic materials because of the possible lack of long-term lateral restraint for the column. Peat lenses are frequently encountered in soft compressible deposits. The thickness and structure of the peak layers are important parameters affecting the use of stone columns. A fibrous peat is considered preferable to non-fibrous peat due to the reinforcement provided by the fibers.

To prevent problems with excessive settlement and stability of the stone column, the ratio of the peat layer thickness to the stone column diameter must always be kept less than two and generally less than one. As previously discussed, when peat is encountered all of the loose organic material must be flushed out of the hole as quickly as possible. Flushing may, however, create a large diameter hole in the peat layer. Stone of 4 in. (100 mm) diameter may be used to form a column through the peat layer although some contractors feel this is unnecessary (refer to Chapter V for another philosophy of stone column construction in peat). The purpose of the large stone is to help bridge the weak peat layer and prevent excessive penetration of stone into the peat. When the peat layer is thick, two and sometimes up to four vibrators are fastened together to form a stone column meeting the required thickness to diameter criterion.

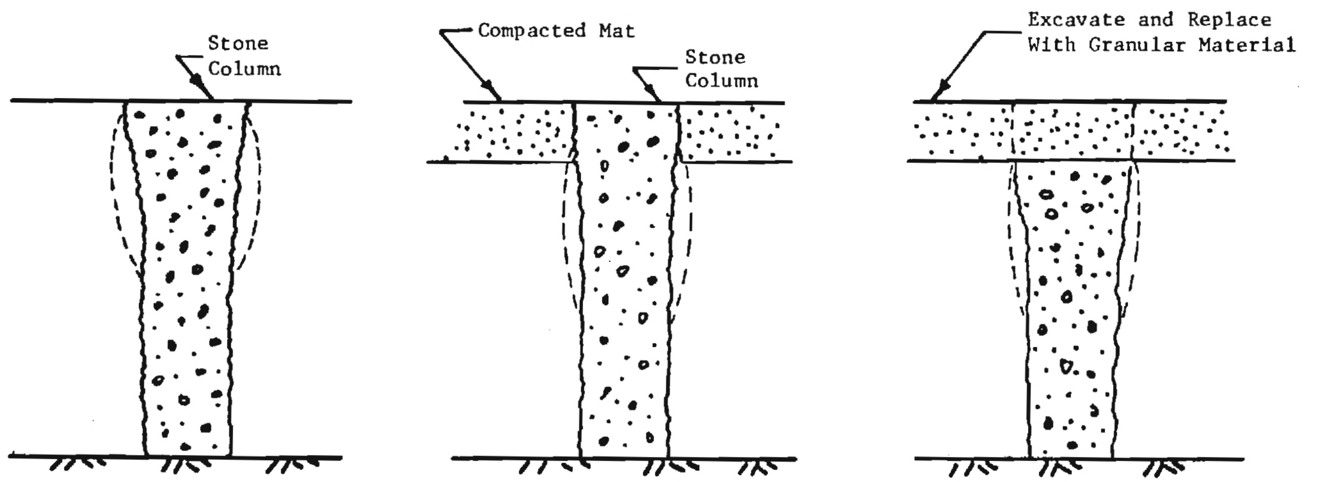
Special consideration must be given to the construction of stone columns in silts and sensitive clays which undergo large strength loss when subjected to vibrations during stone column construction. All contractors indicated that saturated silty soils tend to lose strength during stone column construction (i.e., to be sensitive). Saturated silts lose strength when subjected to vibration due to a build-up in pore pressure. Actual field trials at the site are used to establish the best construction procedures. To minimize the effect of strength loss in either silts or sensitive clays the vibro-replacement (wet) technique should be used, and construction carried out as rapidly as possible. Prolonged compaction of the stone can result in a large diameter, poorly compacted column surrounded by a soil which has undergone significant strength loss due to excessive vibration.

On a site underlain by soft soils and/or having a high groundwater table, an uncompacted mat of granular material should be placed to facilitate construction. The working platform serves a dual purpose by also improving the performance of the stone columns. The granular blanket forces the bulge to a lower depth where the overburden pressure is greater (Fig. 4), and hence results in a larger ultimate capacity of the column. Additionally, the working platform acts as a distribution blanket to help spread the load to the stone columns. The working platform should be about 1 to 3 ft. (0.3-1 m) thick and constructed using sand, gravel or crushed stone. When a working platform is not necessary, a granular blanket is occasionally placed after the columns are constructed. Also, the soil between the constructed columns is sometimes excavated and replaced with the granular material. The granular blanket also serves the important purpose in soft ground projects of an upper drainage layer for the dissipation of pore pressure.

Load tests on single stone columns are often performed at the beginning of the project; load tests on small column groups are performed much less frequently. Although the test should be carried to failure, because of the cost of developing the required reaction, the typical load test is generally carried to 100-150 percent of the design load. On large projects using column groups to support structural load, area load tests should be employed to verify the design load. An area load test typically consists of 5 load increments up to the design load and costs about $7000.

Proof testing of production columns is sometimes performed, particularly in Europe, to verify the workmanship and consistency of construction. *Proof tests should not be considered as an alternative for area load tests.* A proof test consisting of 3 load increments up to a maximum of 20 to 35 tons costs about $1,000 to $1,500 and takes 1 to 2 days to perform. The specific number of proof load tests to insure good workmanship depends, of course, on the size and importance of the job and the subsurface conditions. Usually, British specifications require a minimum of two (2) proof load tests per contract or at a rate of 1 test per 300 columns. One additional proof test is usually performed for each additional 300 columns after the first 300.

Typically the proof test consists of rapidly loading a 2 to 3 ft. (0.6-1 m) square or circular footing placed on top of the column. A crane can be used to provide a reaction of about 10 to 20 tons. GKN Keller in England



(a) Construction of Stone Column Without Compacted Mat.

(b) Construction of Compacted Granular Mat Prior to Construction of Stone Column.

(c) Construction of Stone Column Prior to Construction of Compacted Granular Mat.

FIGURE 4. EFFECT OF COMPACTED GRANULAR WORKING PLATFORM ON BEHAVIOR OF STONE COLUMN.

now use a special H-shaped frame with four, 10 ton reaction weights in performing proof tests to free up use of the expensive crane. Proof tests are actually more a measure of consistency and workmanship than of group bearing capacity. As a rule-of-thumb, Cementation uses a 0.12 to 0.40 in. (3-10 mm) settlement at a load of 11 tons applied to a 2 ft. (0.6 m) diameter plate as an indicator of proper construction. For detailed analysis of this type of test with respect to bearing capacity, the confining effect of the equipment used for the reaction and the effect of bulge of the single column must be evaluated and related to the anticipated prototype conditions (refer to Chapter VII).

Finally, contractors generally feel better subsurface information than is presently made available is needed for fully evaluating the applicability of stone columns for a particular site. To bid intelligently and stay out of trouble, contractors want complete and reliable information describing the subsurface conditions; this is more important for all ground modification methods than for conventional types of deep foundations. Frequently contractors have to base their design on a few widely spaced boring logs.

Specifically the contractors want accurate logging of the test borings, classification and grain size of the subsurface soils together with vane shear, blow count, or dynamic penetrometer test results. The geologic history of the depsoit and the sensitivity of the soil is also necessary. The undrained shear strength, consolidation characteristics, unit weight and water content are often considered necessary. Accurate information is particularly needed giving the occurrence and extent of silt and peat layers. In northern England, where the dry technique is often used on extremely heterogeneous deposits of rubble fill from urban redevelopment, test trenches and pits are often opened prior to bid preparation to allow contractors the opportunity to visually assess the actual conditions. Both engineers and contractors report that this approach is quite effective.

SUMMARY AND CONCLUSIONS

Stone columns have a definite role in the area of ground improvement and stabilization. Vibro-constructed stone columns are best suited for sites consisting of very soft and soft compressible silts and clays, and also for loose silty sands. For economic reasons, the thickness of the strata to be improved should usually be no greater than about 30 ft. (9 m). In general the weak layer should be underlain by a competent bearing strata to realize optimum utility and economy. The design load of stone columns is generally between 20 and 50 tons per column as described in Chapter III and VII.

When properly constructed in suitable soils, stone columns offer a practical alternative to conventional techniques of ground improvement. By replacing a portion of the soft soils with a compacted granular backfill, a composite material is formed which is both stiffer and stronger than the unimproved native soil. Also the subsurface soils, when improved with stone columns, have more uniform strength and compressibility properties than prior to improvement. During the past 20 years specialty contractors have accumulated extensive experience in constructing and testing stone columns.

Stone columns may be constructed by vibro-replacement (wet) process, vibro-displacement (dry) process or less frequently by ramming. In environmentally sensitive areas, stone columns in Europe are frequently constructed by the vibro-displacement (dry) process rather than the vibro-replacement (wet) process which discharges large quantities of silty water. The type equipment and construction procedures used by the various vibro contractors in concept are quite similar, but specific details frequently vary considerably. A thorough subsurface investigation, proper construction technique and adequate inspection are all necessary to assure a satisfactory end product. Important factors in stone column construction include (1) keeping the probe in the hole at all times particularly in soft soils, (2) using a large quantity of water throughout construction, and (3) repenetrating the stone several times by the probe during the construction of each lift.

Subsurface conditions for which stone columns are in general not suited include (1) layers of peat, decomposable organics or refuse greater than 1 to 2 stone column diameters in thickness, (2) sensitive clays and silts which lose their strength when vibrated, and (3) weak strata not underlain by a competent bearing layer. In special cases, even these soils may be improved, but not without extreme care and perhaps great expense. Rigid stone columns offer one solution to some of these limitations. For each ground improvement problem all feasible design alternatives must be thoroughly evaluated before selecting the most cost effective method which will perform satisfactorily.

CHAPTER III

THEORY

INTRODUCTION

Typical applications of stone columns have been described in Chapter I. To economically utilize stone columns to the fullest extent, theories must be available for considering settlement, bearing capacity and general stability for problems involving both single stone columns and stone column groups. In this chapter the failure mechanisms of both a single stone column and a stone column group are first described based on available information. Selected methods are then presented for predicting settlement, bearing capacity and slope stability. Finally, an attempt is made based on limited full-scale test results to relate selected theories to observed field performance. Design recommendations for each mode of failure are given in Chapter VII, and example problems in Appendices C, D, and E.

FAILURE MECHANISMS

Single Stone Columns

Stone columns may be constructed as either end bearing on a firm stratum underlying soft soil, or as floating columns with the tip of the column embedded within the soft layer. In practice however, end bearing stone columns have almost always been used in the past.

Consider a stone column loaded over just the area of the column as shown in Fig. 5. Either end bearing or free floating stone columns greater than about three diameters in length fail in bulging [11] as illustrated in Fig. 5a. A very short column bearing on a firm support will undergo either a general or local bearing capacity type failure at the surface (Fig. 5b). Finally, a floating stone column less than about 2 to 3 diameters in length may fail in end bearing in the weak underlying layer before a bulging failure can develop (Fig. 5c). For the subsurface conditions generally encountered in practice, however, bulging is usually the controlling failure mechanism.

Small scale model studies have shown that the bearing capacity and settlement behavior of a single stone column is significantly influenced by the method of applying the load as shown in Fig. 6. Applying the load through a rigid foundation over an area greater than the stone column (Fig. 6a) increases the vertical and lateral stress in the surrounding soft soil. The larger bearing area together with the additional support of the stone column results in less bulging (Fig. 7) and a greater ultimate load capacity.

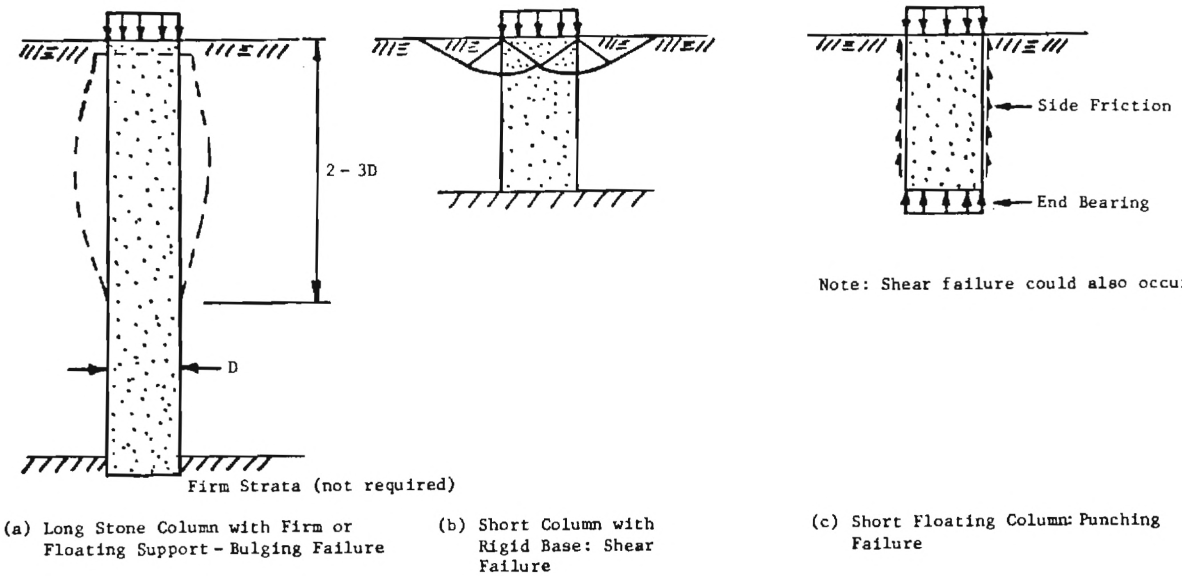


FIGURE 5. FAILURE MECHANISMS OF A SINGLE STONE COLUMN IN A HOMOGENEOUS SOFT LAYER.

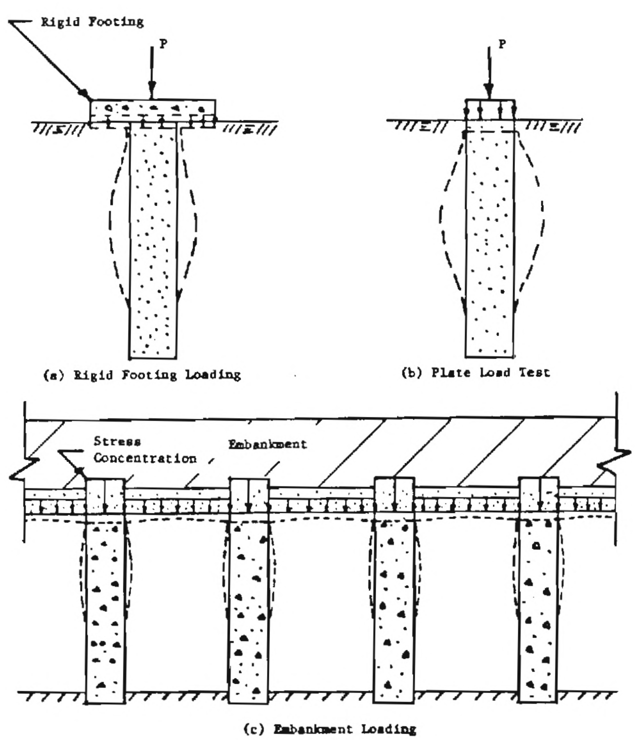


FIGURE 6. DIFFERENT TYPE LOADINGS APPLIED TO STONE COLUMNS.

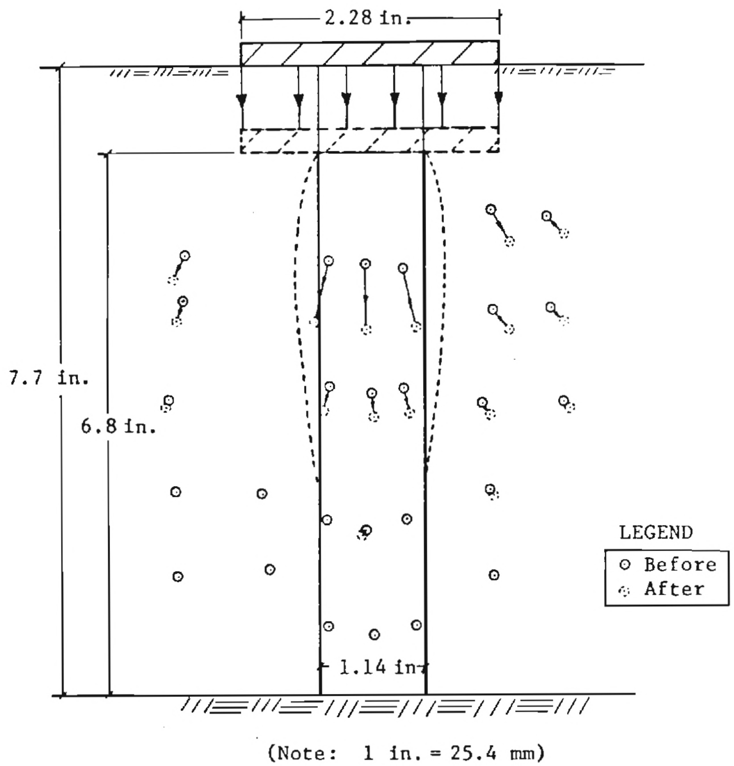


FIGURE 7. DISPLACEMENT VECTORS FOR SINGLE STONE COLUMN LOADED BY RIGID SQUARE PLATE - AREA REPLACEMENT RATIO = 0.20.

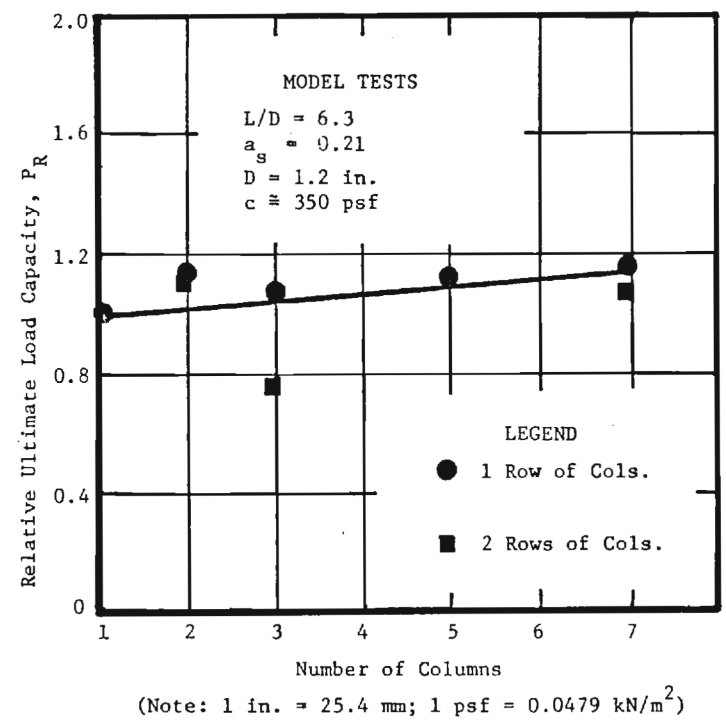


FIGURE 8. INCREASE IN LOAD CAPACITY PER COLUMN WITH INCREASING TOTAL NUMBER OF COLUMNS.

Model tests (Chapter VII) indicate the total ultimate capacity of a square foundation having a total area four times that of the stone column beneath it is about 1.7 times greater than if just the area of the stone column is loaded. For a given load, a stone column loaded by a large rigid plate settles less than if just the stone column is loaded since a portion of the load is carried by both the stone column and the soft clay.

Stone Column Groups

An isolated single column compared to a stone column group has a slightly smaller ultimate load capacity per column than in the group. As surrounding columns are added to form a group, the interior columns are confined and hence somewhat stiffened by the surrounding columns. This results in a slight increase in the ultimate load capacity per column. Small-scale model studies show, for groups having 1 and 2 rows of stone columns, that only a small increase in capacity per column occurs with increasing number of columns (Fig. 8). A rigid foundation loading was used in these tests.

Now consider a wide flexible loading such as an embankment constructed over a stone column improved ground as illustrated in Fig. 6c and 9a. Vautrain [63] has found the settlement of the compressible soil and stone column to be approximately equal beneath an embankment. Due to the construction of the embankment over the weak foundation, the soil beneath and to the sides of the foundation move laterally outward as illustrated in Fig. 9a and 9b. This phenomenon is called "spreading" and has been considered for soft soils not reinforced with stone columns elsewhere [69,70]. Experience and finite element analyses have shown, as would be expected, that settlements are greater when spreading occurs than if spreading is prevented. Compared to the restrained condition, spreading reduces the lateral support given to the stone column and surrounding soil. Lateral spreading also slightly increases the amount of bulging the stone column undergoes compared to the condition of no spreading.

The lateral spreading displacements observed using inclinometers at the Jourdan Road Terminal test embankment [71] located in New Orleans are illustrated in Fig. 10. At this site a small Reinforced Earth retaining wall was supported by 14 stone columns 3.75 ft. (1.1 m) in diameter placed over an area of about 36 ft. (11 m) by 14 ft. (4 m) in plan. Soil surcharge was placed on the reinforced earth wall and then an excavation was made in front of the wall until a rotational stability failure occurred as illustrated in Fig. 11.

A group of stone columns in a soft soil probably undergoes a combined bulging and local bearing type failure as illustrated in Fig. 9c. A local bearing failure is the punching of a relatively rigid stone column (or group) into the surrounding soft soil. Stone column groups having short column lengths can fail in end bearing (Fig. 9d) or perhaps undergo a bearing capacity failure of individual stone columns similar to the failure mode of short, single stone columns.

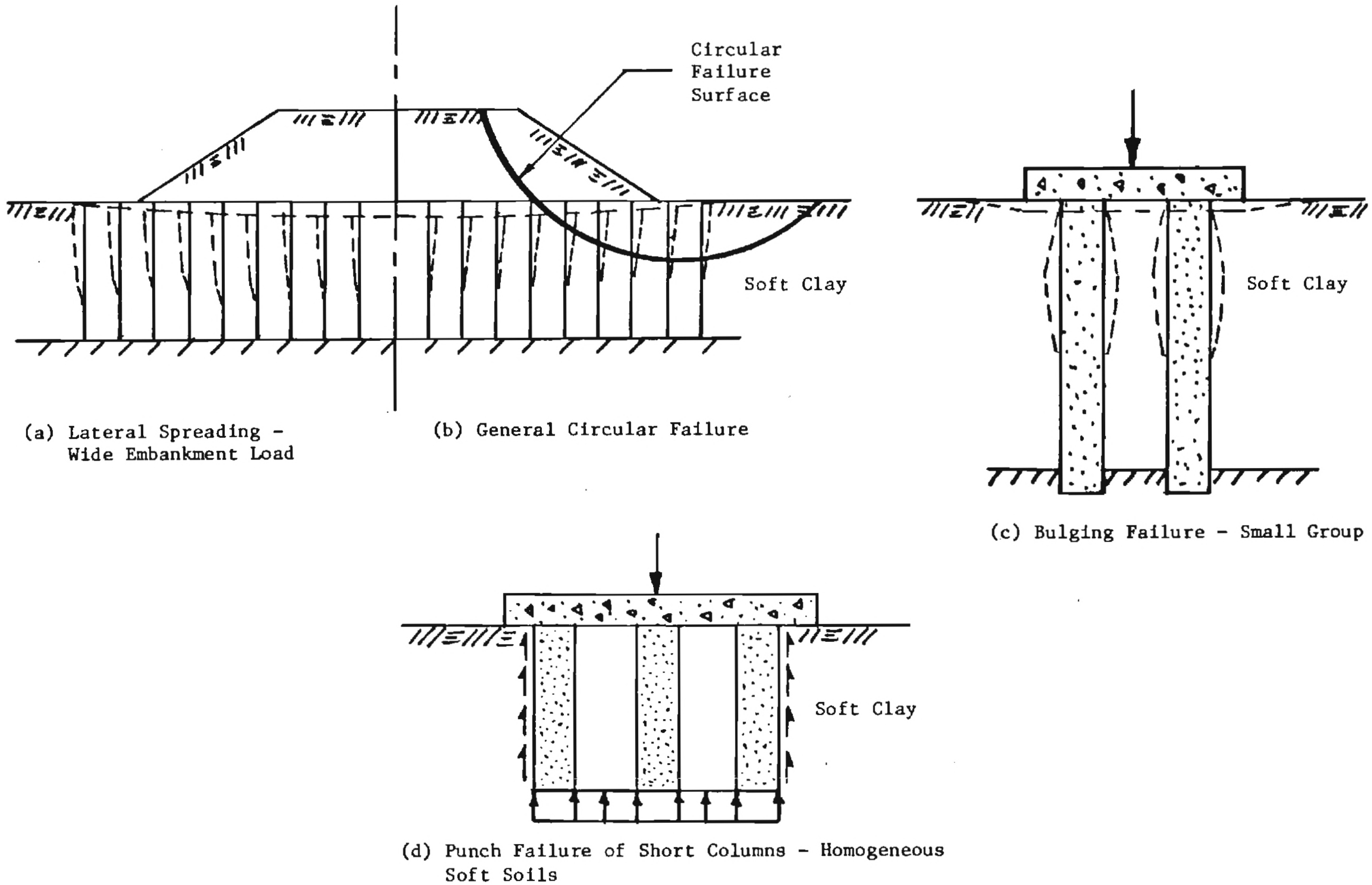


FIGURE 9. FAILURE MODES OF STONE COLUMN GROUPS.

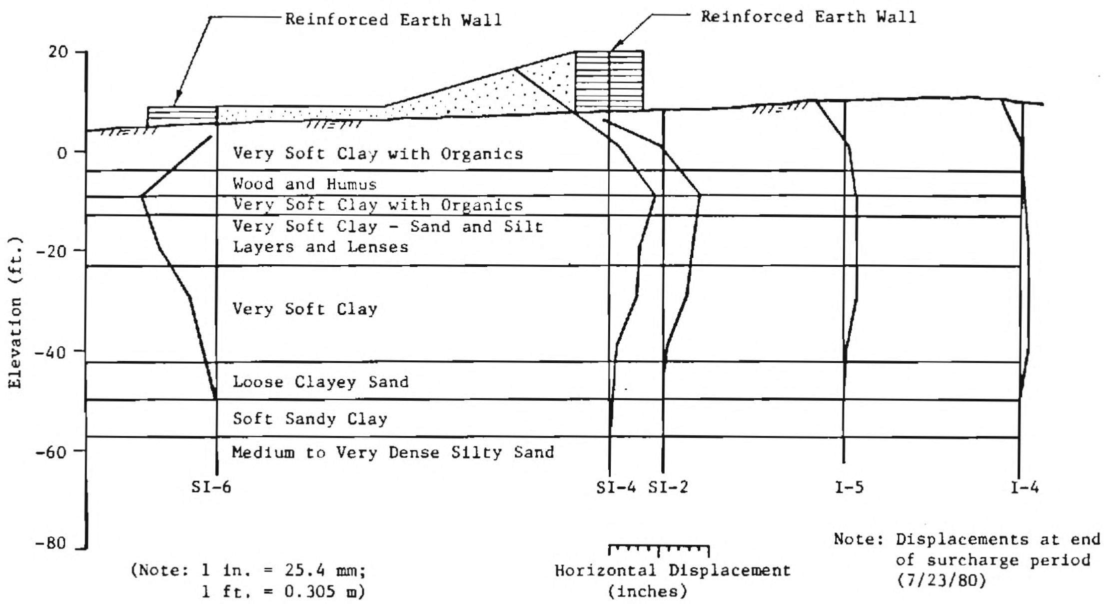


FIGURE 10. LATERAL DISPLACEMENT ALONG EMBANKMENT CENTERLINE AT END OF SURCHARGE PERIOD - JOURDAN ROAD TERMINAL [71].

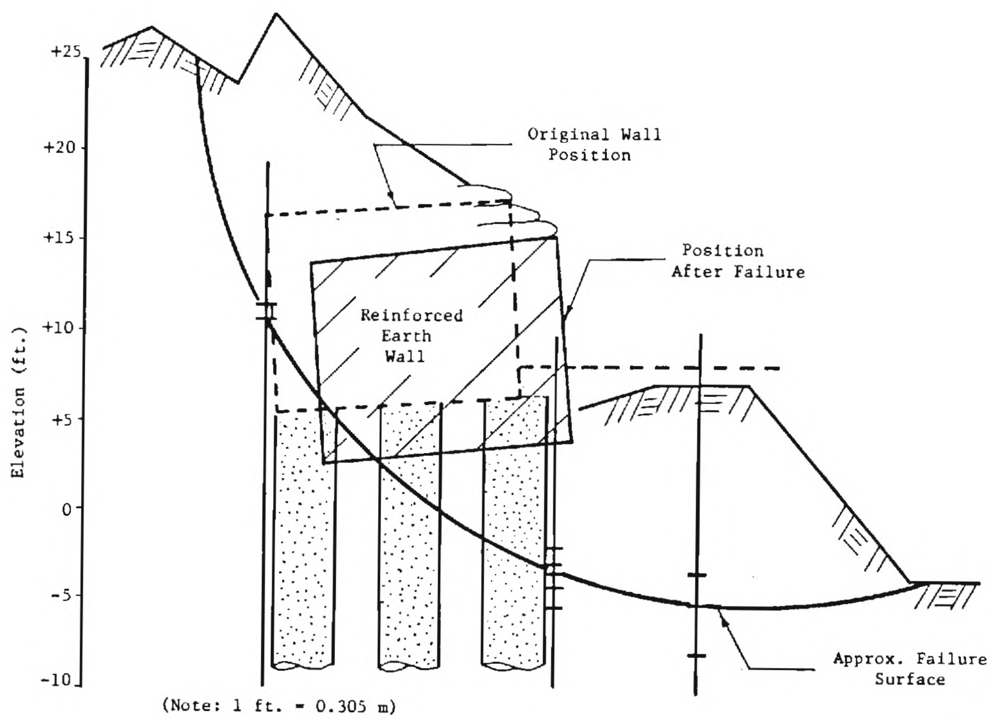


FIGURE 11. JOURDAN ROAD TERMINAL - REINFORCED EARTH EMBANKMENT WITH STONE COLUMNS - COMPOSITE CROSS SECTION AFTER FAILURE [71].

Discussion

The failure mechanisms described above are idealized, assuming uniform soil properties which of course seldom, if ever, are found in nature. Certainly more studies are needed to verify the failure modes of stone column groups. Experience indicates that isolated zones of very soft cohesive soils can result in significant bulging at both shallow and deep depths as illustrated in Fig. 12. A very soft zone at the surface, 3 to 10 ft. (1-3 m) thick, has a dominating influence on the settlement and ultimate strength of either stone column groups or single columns (Fig. 12a). Further, field experience indicates the presence of a very weak layer such as peat greater than about one column diameter in thickness can also seriously affect stone column performance (Fig. 12b and 12c). The lateral deformation pattern observed at the Jourdan Road Terminal test embankment suggests that lateral movements of the stone columns and adjacent soil in a localized zone may have played an important role in the performance of that test embankment.

The failure mechanisms discussed above are based in part on field observations, model tests and finite element studies. Certainly more research in the form of full-scale experiments and model studies are needed to develop detailed knowledge concerning the behavior of stone columns. As discussed later, relatively little is known concerning the interaction between the stone column and surrounding soft soil.

BASIC RELATIONSHIPS

Stone columns are constructed usually in an equilateral triangular pattern although a square pattern is sometimes used. The equilateral triangle pattern gives the most dense packing of stone columns in a given area. A typical layout of stone columns in an equilateral triangular pattern is shown in Fig. 13.

Unit Cell Concept

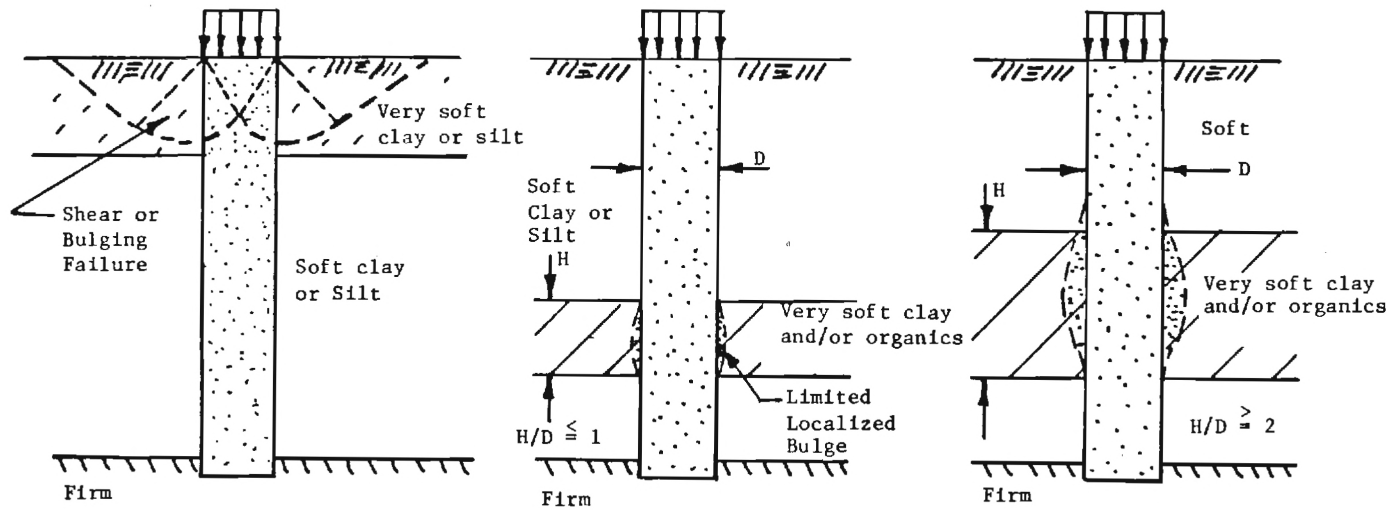
Equivalent Diameter. For purposes of settlement and stability analyses, it is convenient to associate the tributary area of soil surrounding each stone column with the column as illustrated in Figs. 13 and 14. Although the tributary area forms a regular hexagon about the stone column, it can be closely approximated as an equivalent circle having the same total area. For an equilateral triangular pattern of stone columns the equivalent circle has an effective diameter of

$$D_e = 1.05\,s \tag{1}$$

and for a square grid

$$D_e = 1.13\,s \tag{2}$$

where s is the spacing of stone columns. The resulting equivalent cylinder of material having a diameter D_e enclosing the tributary soil and one stone



(a) Soft Layer at Surface - Bulging or Shear Failure

(b) Thin Very Soft Layer - Contained Local Bulge

(c) Thick Very Soft Layer - Local Bulging Failure

FIGURE 12. STONE COLUMN FAILURE MECHANISMS IN NONHOMOGENEOUS COHESIVE SOIL.

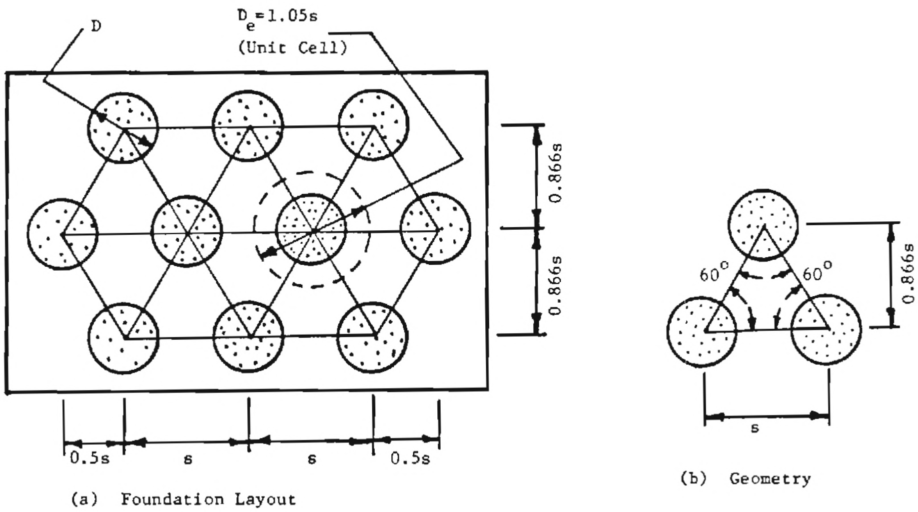


FIGURE 13. EQUILATERAL TRIANGULAR PATTERN OF STONE COLUMNS.

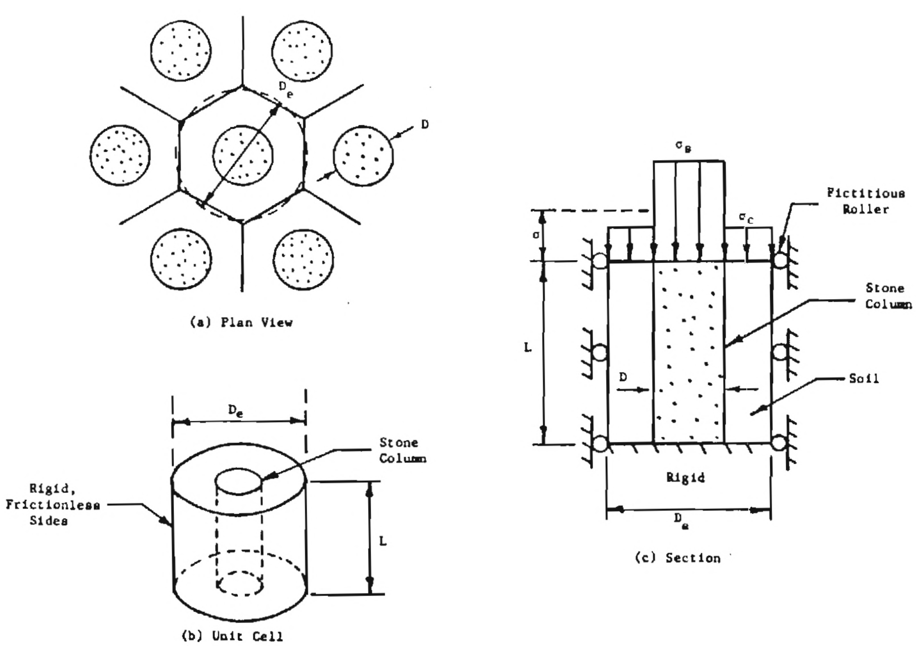


FIGURE 14. UNIT CELL IDEALIZATION.

column is known as the *unit cell.* The stone column is concentric to the exterior boundary of the unit cell (Fig. 14a).

Area Replacement Ratio. The volume of soil replaced by stone columns has an important effect upon the performance of the improved ground. To quantify the amount of soil replacement, define the *Area Replacement Ratio*, a_s, as the fraction of soil tributary to the stone column replaced by the stone:

$$a_s = A_s/A \tag{3}$$

where A_s is the area of the stone column after compaction and A is the total area within the unit cell (Fig. 14a). Further, the ratio of the area of the soil remaining, A_c, to the total area is then

$$\begin{aligned} a_c &= A_c/A \\ &= 1 - a_s \end{aligned} \tag{4}$$

The area replacement ratio, a_s, can be expressed in terms of the diameter and spacing of the stone columns as follows:

$$a_s = C_1 \left(\frac{D}{s}\right)^2 \tag{5a}$$

where: D = diameter of the compacted stone column
s = center to center spacing of the stone columns
C_1 = a constant dependent upon the pattern of stone columns used; for a square pattern $C_1 = \pi/4$ and for an equilateral triangular pattern $C_1 = \pi/(2\sqrt{3})$.

For an equilateral triangular pattern of stone columns the area replacement ratio is then

$$a_s = 0.907\left(\frac{D}{s}\right)^2 \tag{5b}$$

In working with ground improvement using stone columns, it is important to think in terms of the area replacement ratio, a_s.

Extended Unit Cell. Now consider an infinitely large group of stone columns subjected to a uniform loading applied over the area; each interior column may be considered as a unit cell as shown in Figure 14b. Because of symmetry of load and geometry, lateral deformations cannot occur across the boundaries of the unit cell. Also from symmetry of load and geometry the shear stresses on the outside boundaries of the unit cell must be zero. Following these assumptions a uniform loading applied over the top of the unit cell must remain within the unit cell. The distribution of stress within the unit cell between the stone and soil could, however, change with depth. As discussed later, several settlement theories assume this idealized extension of the unit cell concept to be valid. The unit cell can be physically modeled as a cylindrical-shaped container having a frictionless, rigid exterior wall symmetrically located around the stone column (Fig. 14c).

Stress Concentration

Upon placing an embankment or foundation over the stone column reinforced ground, an important concentration of stress occurs in the stone column (Fig. 14c), and an accompanying reduction in stress occurs in the surrounding less stiff soil [19,24,27,39]. Since the vertical settlement of the stone column and surrounding soil is approximately the same [63], stress concentration occurs in the stone column since it is stiffer than a cohesive or a loose cohesionless soil.

Now consider conditions for which the unit cell concept is valid such as a reasonably wide, relatively uniform loading applied to a group of stone columns having either a square or equilateral triangular pattern. The distribution of vertical stress within a unit cell (Fig. 14c) can be expressed by a stress concentration factor n defined as

$$n = \sigma_s/\sigma_c \tag{6}$$

where: σ_s = stress in the stone column
σ_c = stress in the surrounding cohesive soil

The average stress σ which must exist over the unit cell area at a given depth must, for equilibrium of vertical forces to exist within the unit cell, equal for a given area replacement ratio, a_s

$$\sigma = \sigma_s \cdot a_s + \sigma_c(1 - a_s) \tag{7}$$

where all the terms have been previously defined. Solving equation (7) for the stress in the clay and stone using the stress concentration factor n gives [24,66]

$$\sigma_c = \sigma/[1 + (n - 1)a_s] = \mu_c\sigma \tag{8a}$$

and

$$\sigma_s = n\sigma/[1 + (n - 1)a_s] = \mu_s\sigma \tag{8b}$$

where μ_c and μ_s are the ratio of stresses in the clay and stone, respectively, to the average stress σ over the tributary area. For a given set of field conditions, the stress in the stone and clay can be readily determined using equations (8a) and (8b) if a reasonable value of the stress concentration factor is assumed based on previous measurements. The above σ, σ_c and σ_s stresses are due to the applied loading. In addition, the initial effective (and total) overburden and initial lateral stress at a given depth are also important quantities.

The above two equations, which give the stress due to the applied loading in the stone column and surrounding soil, are extremely useful in both settlement and stability analyses. The assumptions made in the derivation of these equations are (1) the extended unit cell concept is valid,

(2) statics is satisfied, and (3) the value of stress concentration is either known or can be estimated. Even where the extended unit cell concept is obviously not valid, use of equations (8a) and (8b) in settlement calculations appears to give satisfactory results, probably because the average change in vertical stress with horizontal distance is not too great. As the number of stone columns in the group decreases, the accuracy of this approach would be expected to also decrease.

ULTIMATE LOAD ANALYSIS

Single Isolated Stone Column

Since most constructed stone columns have length to diameter ratios equal to or greater than 4 to 6, a bulging failure usually develops (Fig. 5a) whether the tip of the column is floating in soft soil or resting on a firm bearing layer. Fig. 15 illustrates the bulging failure of a single model stone column floating in soft clay observed by Hughes and Withers [11]. The bulge that developed occurred over a depth of 2 to 3 diameters beneath the surface. These small-scale model tests were performed using 0.5 in. to 1.5 in. (12.5 to 38 mm) diameter sand columns which were 5.9 in. (150 mm) in length. A soft kaolin clay was used having a shear strength of 400 psf (19.1 kN/m^2). Strains were determined in the composite soil mass from displacements obtained using radiographs taken of lead markers.

As early as 1835, Moreau (referenced by Hughes and Withers) observed that very little of the applied load reaches the bottom of a single column if the column length is greater than twice its width. The fact that load applied to a single stone column is transferred to the surrounding soft soil was verified in the small-scale experiments of Hughes and Withers [11]. As the column simultaneously bulges and moves downward, the granular material presses into the surrounding soft soil [25] and transfers stress to the soil through shear. Theoretical finite element studies indicate near the failure load slippage at the interface between the stone and clay may occur at the top of the column [40,48]. Also, failure of the stone column and surrounding soil occurs early during loading, extending from the surface downward with increasing load.

A number of theories have been presented for predicting the ultimate capacity of an isolated, single stone column surrounded by a soft soil [11,12,14,18,24,29,33,37,48,52-57]. Most of the early analytical solutions assume a triaxial state of stress exists in the stone column, and both the column and surrounding soil are at failure [11,12,24,29,33,52-56].

The lateral confining stress σ_3 which supports the stone column is usually taken in these methods as the ultimate passive resistance which the surrounding soil can mobilize as the stone column bulges outward against the soil. Since the column is assumed to be in a state of failure, the ultimate vertical stress, σ_1, which the column can take is equal to the coefficient of passive pressure of the stone column, K_p, times the lateral confining stress, σ_3, which from classical plasticity theory can be expressed as:

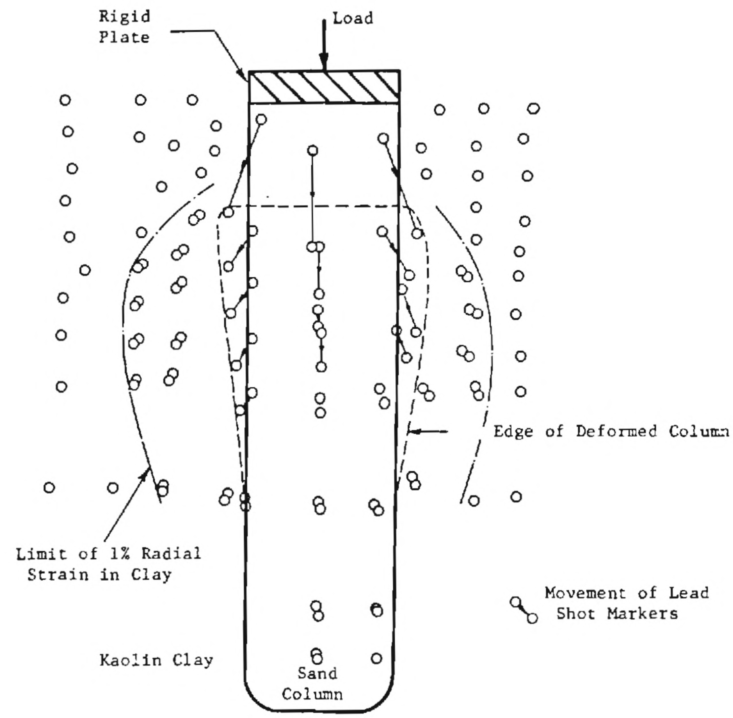


FIGURE 15. BULGING FAILURE MODE OBSERVED IN MODEL TESTS FOR A SINGLE STONE COLUMN LOADED WITH A RIGID PLATE OVER THE COLUMN [11].

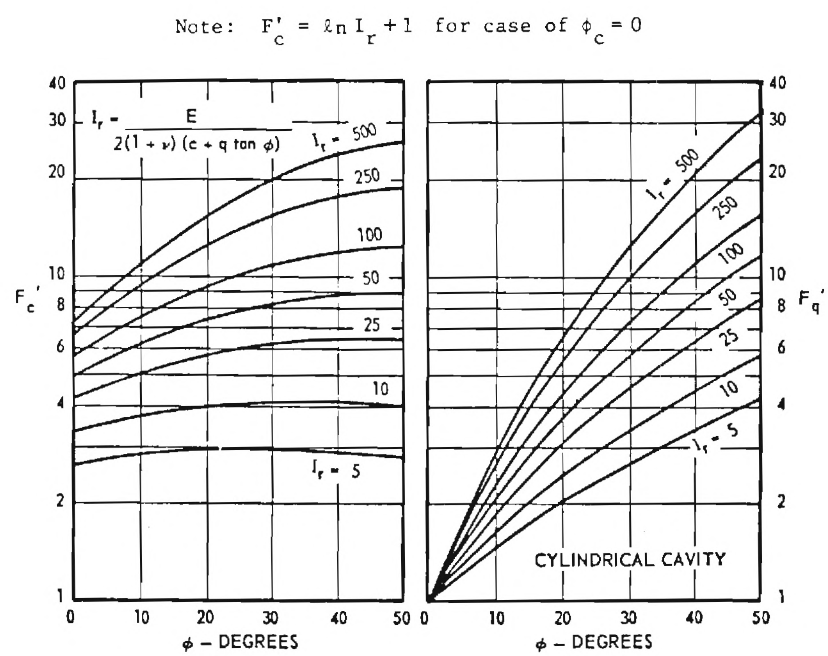


FIGURE 16. VESIC CYLINDRICAL CAVITY EXPANSION FACTORS [61].

$$\sigma_1/\sigma_3 = \frac{1+\sin\phi_s}{1-\sin\phi_s} \tag{9}$$

where ϕ_s = angle of internal friction of the stone column and the stress ratio σ_1/σ_3 is the coefficient of passive earth pressure K_p for the stone column. Finite element analyses indicate the above equation is a good approximation.

Greenwood [19] and later Wong [12] have assumed for preliminary analyses that the lateral resistance the surrounding soil can develop is equal to the passive resistance mobilized behind a long retaining wall which is laterally translated into the soil. Such an approach assumes a plane strain loading condition and hence does not realistically consider the three-dimensional geometry of a single column. The design approach of Wong [12] in its final form does, however, appear to give reasonably good correlation with the measured response of stone column groups.

Cavity Expansion Theory. The passive resistance developed by the surrounding soil as a first approximation can be better modeled as an infinitely long cylinder which expands about the axis of symmetry until the ultimate passive resistance of the surrounding soil is developed. The expanding cylindrical cavity approximately simulates the lateral bulging of the column into the surrounding soil. Hughes and Withers [11], Datye, et al. [52-55] and Walleys, et al. [50,51] have evaluated the confining pressure on the stone column using this approach. Even though the stone column bulges outward along a distance of only 2 to 3 diameters, the model of an infinitely long expanding cylinder appears to give, as an engineering approximation, reasonably good results [11,47].

Hughes and Withers [11] considered the bulging type failure of a single stone column to be similar to the cavity developed during a pressuremeter test. In their approach the elastic-plastic theory given by Gibson and Anderson [64] for a frictionless material and an infinitely long expanding cylindrical cavity was used for predicting undrained, ultimate lateral stress σ_3 of the soil surrounding the stone column:

$$\sigma_3 = \sigma_{ro} + c[1 + \ell n_e \frac{E_c}{2c(1+\nu)}] \tag{10}$$

where: σ_3 = the ultimate undrained lateral stress
σ_{ro} = total in-situ lateral stress (initial)
E_c = elastic modulus of the soil
c = undrained shear strength
ν = Poisson's Ratio

Substituting equation (10) which gives the confining pressure on the stone column into (9) and letting q_{ult} equal σ_1 gives:

$$q_{ult} = \{\sigma_{ro} + c[1 + \log_e \frac{E}{2c(1+\nu)}]\}\left(\frac{1+\sin\phi_s}{1-\sin\phi_s}\right) \tag{11}$$

where q_{ult} is the ultimate stress that can be applied to the stone column. The undrained modulus of elasticity of soft cohesive soils can as an approximation be taken to be proportional to the undrained shear strength.

Vesic Cavity Expansion Theory. Vesic [61] has developed a general cylindrical cavity expansion solution extending earlier work to include soils with both friction and cohesion. Once again the cylinder is assumed to be infinitely long and the soil either elastic or plastic. The effect of volume change in the plastic zone, which tends to reduce the ultimate capacity, can be included in the solution but is not presented here. The ultimate lateral resistance σ_3 developed by the surrounding soil can be expressed as

$$\sigma_3 = c\,F'_c + q\,F'_q \tag{12}$$

where: c = cohesion
q = mean (isotropic) stress $(\sigma_1+\sigma_2+\sigma_3)/3$ at the equivalent failure depth
F'_c, F'_q = cavity expansion factors

The cavity expansion factors F'_c and F'_q shown in Fig. 16 are a function of the angle of internal friction of the surrounding soil and the Rigidity Index, I_r. The Rigidity Index, not reduced for the effects of volume change in the plastic zone, is expressed as

$$I_r = \frac{E}{2(1+\nu)(c+q\tan\phi_c)} \tag{13}$$

where: E = modulus of elasticity of the surrounding soil in which cavity expansion is occurring
c = cohesion of the surrounding soil
ν = Poisson's ratio of the surrounding soil
q = mean stress within the zone of failure

Upon substituting equation (12) into equation (9) and letting q_{ult} equal σ_1, the ultimate stress that can be applied to the stone column becomes:

$$q_{ult} = [c\,F'_c + q\,F'_q]\left(\frac{1+\sin\phi_s}{1-\sin\phi_s}\right) \tag{14}$$

where all the terms have been previously defined.

The general solution developed by Vesic gives, for a frictionless soil, the same ultimate load as the cavity expansion solution of Gibson and Anderson. The mean stress q used in the above analyses should be taken as the stress occurring at the average depth of the bulge. The mean stress q is the sum of both initial stresses existing in the ground and the change in stress due to the externally applied load. Due to stress concentration in the stone column, however, the stress increase in the soil due to external loading will usually be only a portion of q. Both the short and long-term ultimate capacity of a stone column can be estimated using cavity expansion theory. Also, the increase in strength of the soft soil should be considered due to preloading and/or consolidation which occurs during construction.

Short Stone Columns. A short stone column may fail either by a general or local bearing capacity failure of the stone and surrounding soil (Fig. 5b), or else by punching into a soft underlying soil (Fig. 5c). The ultimate capacity for a punching failure can be determined by calculating the end bearing capacity of the stone column using conventional bearing capacity theories and adding the skin friction load developed along the side of the column.

A general bearing capacity failure could occur at the surface where the overburden surcharge effect is the smallest. Madhav and Vitkar [38] have presented the plane strain solution for a general bearing capacity failure of a trench filled with granular material constructed in a frictionless soil. The solution utilizes the upper bound limit analysis theorems of Drucker and Prager. As shown in Fig. 17, the loading may be applied to both the granular stone and the adjacent soft clay. From their solution the ultimate bearing capacity is given for a plane strain loading as

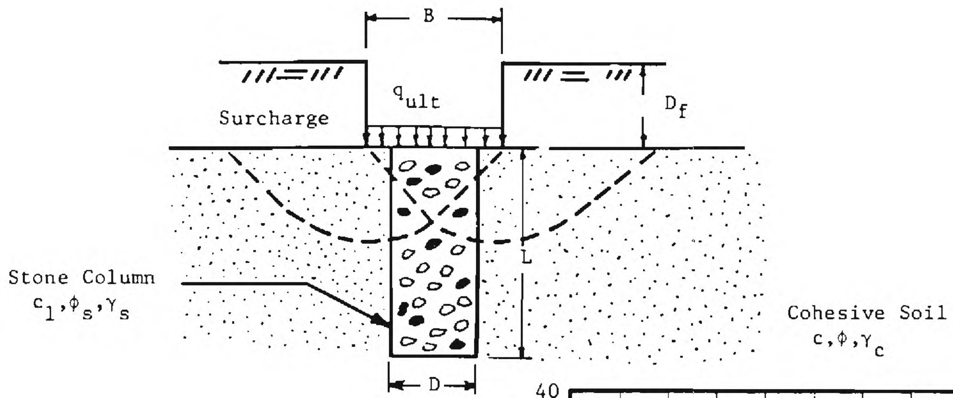
$$q_{ult} = \frac{\gamma_c B}{2} N_\gamma + c\, N_c + D_f \gamma_c N_q \tag{15}$$

where N_γ, N_c, and N_q are bearing capacity factors given in Fig. 17, and the other terms used in the equation are also defined in the figure. An approximate solution for the axisymmetric loading condition can be obtained by correcting the bearing capacity factors using the shape factors recommended by Vesic [65].

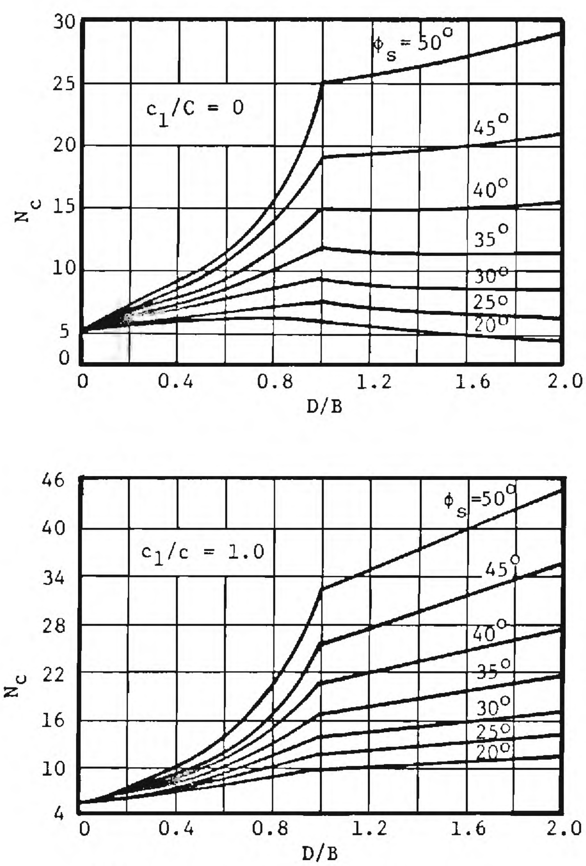
Ultimate Capacity of Stone Column Groups

Consider the ultimate strength of either a square or infinitely long, rigid concrete footing resting on the surface of a cohesive soil reinforced with stone columns as illustrated in Fig. 18. Assume the foundation is loaded quickly so that the undrained shear strength is developed in the cohesive soil, with the angle of internal friction being negligible. Also neglect cohesion in the stone column. Finally, assume, for now, the full shear strength of both the stone column and cohesive soil is mobilized. The ultimate bearing capacity of the group can be determined by approximating the failure surface by two straight rupture lines. Such a theory was first developed for homogeneous soils by Bell and modified by Terzaghi and Sowers [74]. For homogeneous soils, this theory compares favorably with the Bell bearing capacity theory and gives results reasonably close to the Terzaghi local bearing failure theory.

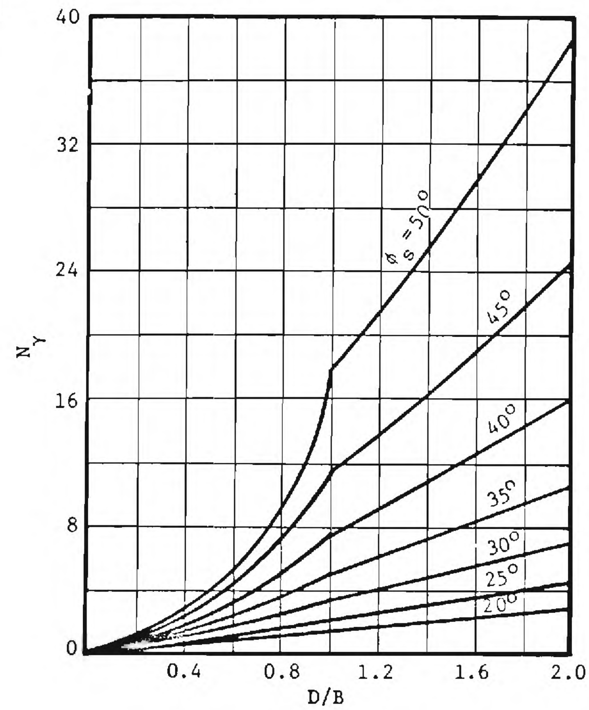
Assume as an approximation that the soil immediately beneath the foundation fails on a straight rupture surface, forming a triangular block



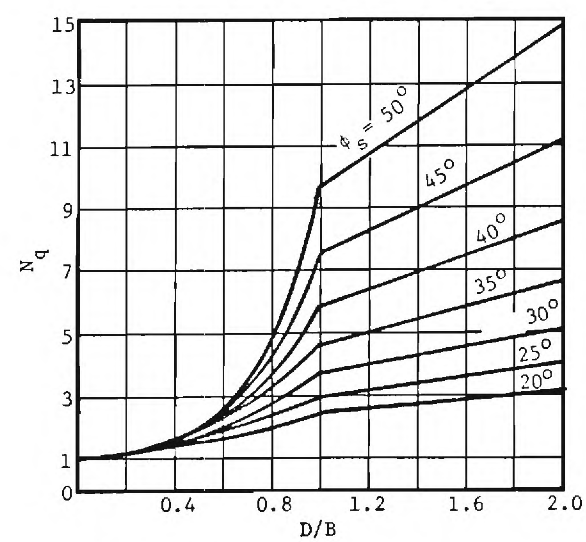
For Case Shown: $\gamma_s/\gamma_c = 1.0$
$\phi = 0$



(a) Bearing Capacity Factor, N_c



(b) Bearing Capacity Factor, N_γ



(c) Bearing Capacity Factor, N_q

FIGURE 17. GENERAL BEARING CAPACITY FAILURE FOR STRIP LOAD AND STONE COLUMN - PLANE STRAIN [38].

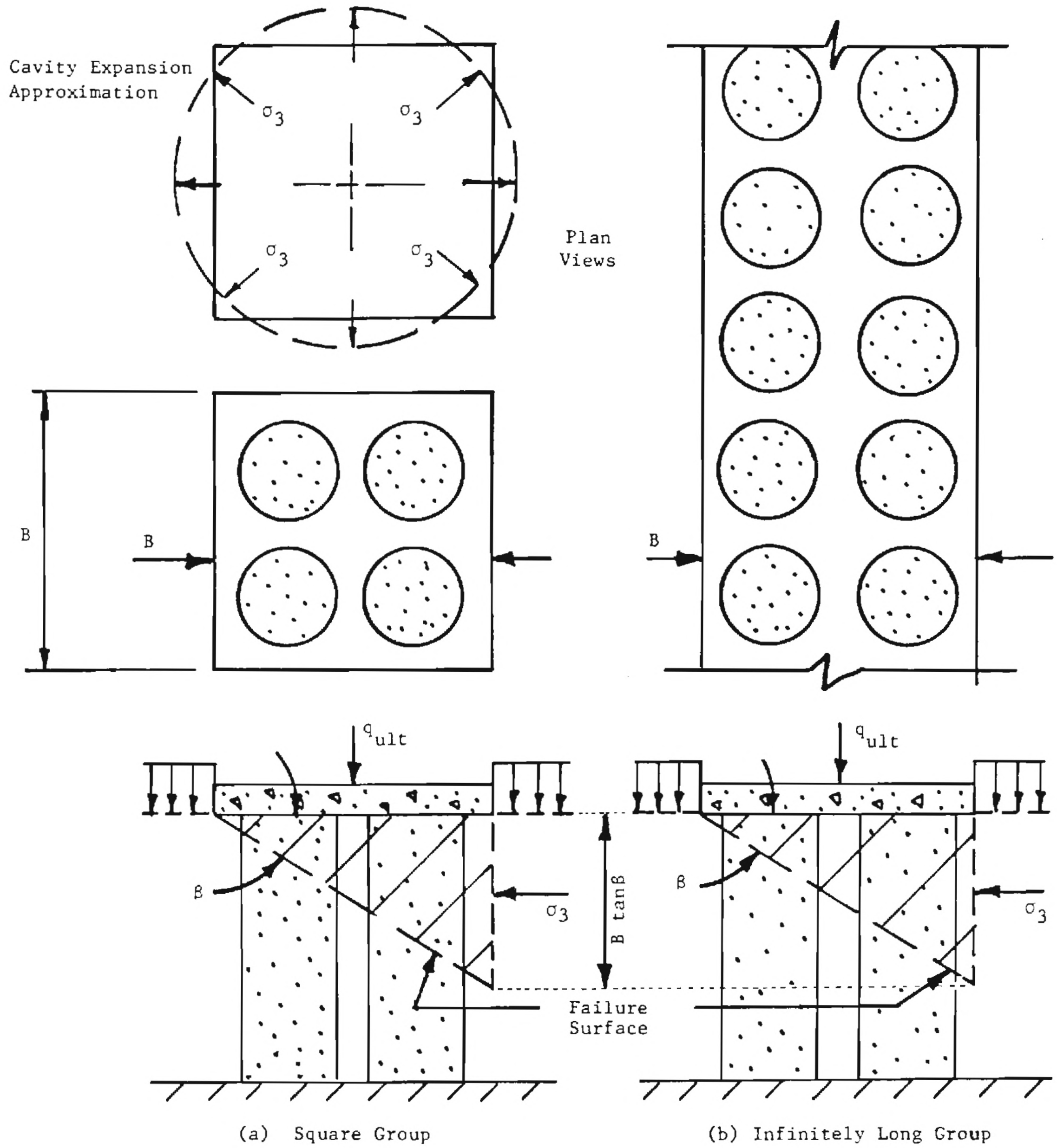


FIGURE 18. STONE COLUMN GROUP ANALYSIS - FIRM TO STIFF COHESIVE SOIL.

as shown in Figure 18. The average shear resistance of the composite soil would be developed on the failure surface. The ultimate stress q_{ult} that the composite soil can withstand is dependent upon the lateral, ultimate resistance σ_3 of the block to movement and the composite shear resistance developed along the inclined shear surface. From a consideration of equilibrium of the block the average shear strength parameters within the block are

$$[\tan\phi]_{avg} = \mu_s a_s \tan\phi_s \tag{16a}$$

and

$$c_{avg} = (1 - a_s)c \tag{16b}$$

where $[\tan\phi]_{avg}$ is the tangent of the composite angle of internal friction and c_{avg} is the composite cohesion on the shear surface beneath the foundation; a_s is the area replacement ratio and μ_s is the stress concentration factor for the stone, as defined by equations (3) and (8b) respectively. As mentioned previously, the strength components due to cohesion of the stone and friction of the clay are neglected in this derivation. The failure surface makes an angle β with the foundation, where β for the composite soil is

$$\beta = 45 + \frac{\phi_{avg}}{2} \tag{17}$$

and

$$\phi_{avg} = \tan^{-1} (\mu_s a_s \tan\phi_s)$$

To calculate the ultimate capacity for a group first determine the ultimate lateral pressure σ_3. For an infinitely long footing from classical earth pressure theory for a saturated clay having only cohesion c:

$$\sigma_3 = \frac{\gamma_c \; B \tan\beta}{2} + 2c \tag{18}$$

where:
- σ_3 = average lateral confining pressure
- γ_c = saturated or wet unit weight of the cohesive soil
- B = foundation width
- β = inclination of the failure surface as given by equation (17)
- c = undrained shear strength within the unreinforced cohesive soil

The lateral confining pressure for a square foundation can be determined using the cavity expansion theory of Vesic, equation (12). The Vesic cylindrical expansion theory gives the ultimate stress that can be exerted on the failure block by the surrounding soil. The three-dimensional failure on a cylindrical surface should give a satisfactory approximation of the three-dimensional failure of a square foundation.

Assuming the ultimate vertical stress q_{ult} (which is also assumed to be

σ_1) and ultimate lateral stress σ_3 to be principal stresses, equilibrium of the wedge requires

$$q_{ult} = \sigma_3 \tan^2\beta + 2c_{avg} \tan\beta \tag{19}$$

where σ_3 is obtained from equation (18) and the other terms have been previously defined. The effect of soil weight within the wedge was conservatively neglected. The soil weight within the wedge would increase the composite shear resistance and could be included in the analysis by appropriately modifying equation (16a). Such a degree of refinement is not considered justified.

The proposed method for estimating the ultimate capacity of stone column groups considers (1) foundation shape, (2) foundation size, (3) the angle of internal friction of the stone column, (4) composite shear strength of the stone column reinforced soil, (5) the shear strength and overburden pressure in the soil surrounding the foundation, and (6) the compressibility of the surrounding soil as defined by the Rigidity Index, equation (13). In applying this approach it must be remembered that the composite strength of the stone column reinforced soil below the foundation is considered mobilized; therefore in soft soils use of a composite strength which is less than the combined individual strengths of the two materials at failure is required to reflect the actual shear resistance mobilized along the failure wedge (refer to Chapter VII).

Unit Cell Idealization. As one bound, a large stone column group can be approximated as an infinitely large group of columns. A stone column and its tributary soil located on the interior of the infinite array can be theoretically modeled using the unit cell concept. Since within a large group of stone columns the settlement of the soil and stone column is approximately equal, a rigid plate loading on the top of the unit cell can, as an approximation, be visualized. The model of a unit cell loaded by a rigid plate is analogous to a one-dimensional consolidation test. In this test a bearing capacity failure does not occur since loading is along the K_o stress path line. Indeed, consolidation tests performed on unit cell models as a part of this study and also large scale tests at the Building Research Establishment [124] in England both showed similar performance to a consolidation test with failure not occurring. For stone column groups used in practice which are always of limited size, however, it is not likely that the unit cell condition of infinite boundary rigidity would ever be developed due to lateral spreading and bulging. In practical applications both lateral deformations of the stone column and spreading in the direction of least lateral resistance, lateral consolidation of the soil surrounding the stone column, and the presence of locally very soft zones are all encountered. The ultimate load capacity is therefore limited to a finite value slightly larger than for a single column (Fig. 8).

SETTLEMENT

Presently available methods for calculating settlement can be classified as either (1) simple, approximate methods which make important simplifying assumptions or (2) sophisticated methods based on fundamental elasticity and/or plasticity theory (such as finite elements) which model material and boundary conditions. Several of the more commonly used approximate methods are presented first. Following this, a review is given of selected theoretically sophisticated elastic and elastic-plastic methods and design charts are presented. All of these approaches for estimating settlement assume an infinitely wide, loaded area reinforced with stone columns having a constant diameter and spacing. For this condition of loading and geometry the extended unit cell concept is theoretically valid and has been used by the Japanese [24,66], Priebe [14], and Goughnour, et al. [33] and in the finite element method to develop theoretical solutions for predicting settlement. As discussed in the next major section, the reduction in settlement can be approximately considered due to the spreading of stress in groups of limited size.

Equilibrium Method

The equilibrium method described for example by Aboshi, et al. [24] and Barksdale [66] is the method used in Japanese practice for estimating the settlement of sand compaction piles. The equilibrium method also offers a very simple yet realistic engineering approach for estimating the reduction in settlement of ground improved with stone columns. In applying this simple approach the stress concentration factor, n, must be estimated using past experience and the results of previous field measurements of stress. A discussion of measured stress concentration factors is given in Chapter VII. If a conservatively low stress concentration factor is used, a safe estimate of the reduction in settlement due to ground improvement will be obtained.

The following assumptions are necessary in developing the equilibrium method: (1) the extended unit cell idealization is valid, (2) the total vertical load applied to the unit cell equals the sum of the force carried by the stone and the soil (i.e., equilibrium is maintained within the unit cell), (3) the vertical displacement of stone column and soil is equal, and (4) a uniform vertical stress due to external loading exists throughout the length of stone column, or else the compressible layer is divided into increments and the settlement of each increment is calculated using the average stress increase in the increment. Following this approach, as well as the other methods, settlements occurring below the stone column reinforced ground must be considered separately; usually these settlements are small and can often be neglected.

The change in vertical stress in the clay, σ_c, due to the applied external stress is equal to

$$\sigma_c = \mu_c \sigma$$

where σ is the average externally applied stress (Fig. 14c), and μ_c is given by equation (8a). From conventional one-dimensional consolidation theory

$$S_t = \left(\frac{C_c}{1+e_o}\right) \log_{10}\left(\frac{\bar{\sigma}_o + \sigma_c}{\bar{\sigma}_o}\right) \cdot H \tag{20}$$

where: S_t = primary consolidation settlement occurring over a distance H of stone column treated ground

H = vertical height of stone column treated ground over which settlements are being calculated

$\bar{\sigma}_o$ = average initial effective stress in the clay layer

σ_c = change in stress in the clay layer due to the externally applied loading, equation (8a)

C_c = compression index from one-dimensional consolidation test

e_o = initial void ratio

Ground Improvement. From equation (20) it follows that for normally consolidated clays, the ratio of settlements of the stone column improved ground to the unimproved ground, S_t/S, can be expressed as

$$S_t/S = \frac{\log_{10}\left(\frac{\bar{\sigma}_o + \mu_c\sigma}{\bar{\sigma}_o}\right)}{\log_{10}\left(\frac{\bar{\sigma}_o + \sigma}{\bar{\sigma}_o}\right)} \tag{21}$$

This equation shows that the level of improvement is dependent upon (1) the stress concentration factor n (as reflected in μ_c), (2) the initial effective stress in the clay, and (3) the magnitude of applied stress σ. Equation (21) indicates, if other factors are constant, a greater reduction in settlement is achieved for longer columns (the average $\bar{\sigma}_o$ increases with stone column length) and for smaller applied stress increments.

For very large $\bar{\sigma}_o$ (long length of stone column) and very small applied stresses σ, the settlement ratio relatively rapidly approaches

$$S_t/S = 1/[1+(n-1)\,a_s] = \mu_c \tag{22}$$

where all terms have been previously defined. Equation (22) is shown graphically in Fig. 19; it gives a slightly unconservative estimate of expected ground improvement and is useful for preliminary studies.

Stress Concentration. The stress concentration factor n required to calculate σ_c is usually estimated from the results of stress measurements made for full-scale embankments (refer to Chapter VII), but could be estimated from theory. In some cases the stress concentration factor has been estimated from elastic theory assuming equal vertical displacements of the stone column and surrounding soil. From elastic theory assuming a constant vertical stress, the vertical settlement of the stone column can be approximately calculated as follows:

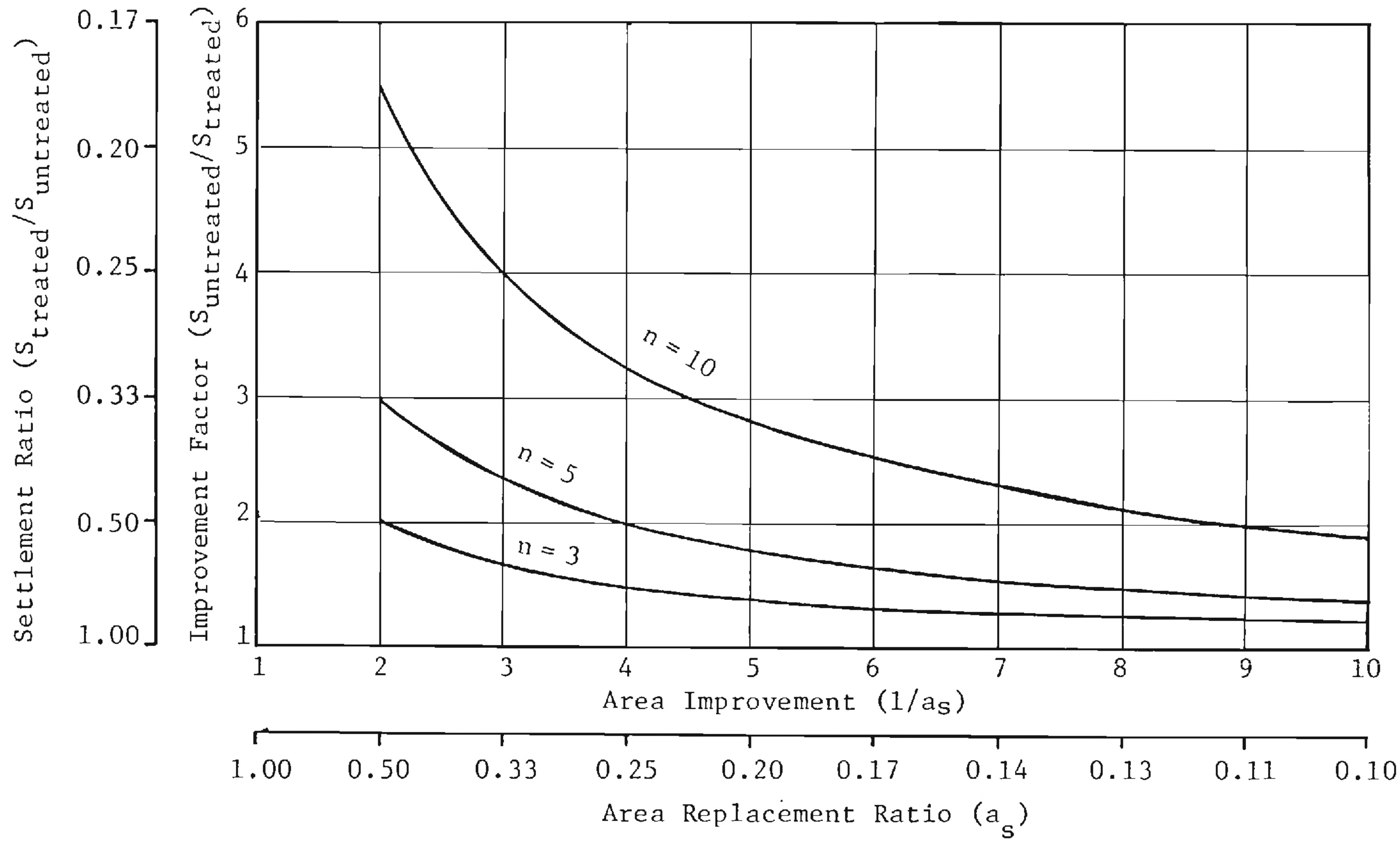


FIGURE 19. MAXIMUM REDUCTION IN SETTLEMENT THAT CAN BE OBTAINED USING STONE COLUMNS - EQUILIBRIUM METHOD OF ANALYSIS.

$$S_s = \frac{\sigma_s L}{\tilde{D}_s} \qquad (23)$$

where: S_s = vertical displacement of the stone column

σ_s = average stress in the stone column

L = length of the stone column

$\tilde{D}_s$ = constrained modulus of the stone column (the elastic modulus, E_s, could be used for an upper bound)

Now assume constant vertical settlement of the stone column and the tributary soil. Using equation (23) and its analogous form for the soil, equate the settlement of the stone and soil to obtain

$$\sigma_s/\sigma_c = \tilde{D}_s/\tilde{D}_c \qquad (24)$$

where σ_s and σ_c are the stresses in the stone column and soil, respectively and $\tilde{D}_s$ and $\tilde{D}_c$ are the appropriate moduli of the two materials. Note that if the constrained moduli of the two materials are used, the stress concentration σ_s/σ_c is also a function of the Poisson's ratio of the two materials. Equation (48), presented later in the discussion section of this chapter, gives the constrained modulus as a function of the modulus of elasticity and Poisson's ratio.

Use of equation (24) gives values of the stress concentration factor n from 25 to over 500 which is considerably higher than measured in the field. Field measurements for stone columns have shown n to generally be in the range of 2 to 5 [27,63]. Therefore, use of the approximate compatibility method, equation (24), for estimating the stress concentration factor is not recommended for soft clays.

Conclusion. *Because of its simplicity, versatility and reasonably good assumptions made in its derivation, the equilibrium method summarized by equation (21) offers a practical approach for estimating settlement reduction due to ground improvement with stone columns.*

Priebe Method

The method proposed by Priebe [14] for estimating reduction in settlement due to ground improvement with stone columns also uses the unit cell model. The stone column is assumed to be in a state of plastic equilibrium under a triaxial stress state. The soil within the unit cell is idealized as an elastic material. Since the stone column is assumed to be incompressible, the change in volume within the soil is directly related to vertical shortening of the cylindrical column which forms the basis of the derivation. The radial deformation of the elastic soil is determined using an infinitely long, elastic hollow cylinder solution. The elastic cylinder of soil, which has a rigid exterior boundary coinciding with the boundary of the unit cell, is subjected to a uniform internal pressure. Other assumptions made in the analysis include (1) equal vertical settlement of the stone and soil, (2) uniform stresses in the two materials, and (3) end bearing on a rigid layer. This approach, as applied in practice, is described elsewhere [75].

The design relationship developed by Priebe is given in Fig. 20. The ratio of settlement of untreated to treated ground S/S_t is given as a function of the area replacement ratio a_s and angle of internal friction of the stone, ϕ_s. Superimposed on these curves for comparison is the upper bound (maximum amount of ground improvement) equilibrium method solution (equation 22) for stress concentration factors of n = 3,5, and 10. The Priebe curves, which are used by GKN Keller, generally fall between the upper bound equilibrium curves for n between 5 and 10. The Priebe improvement factors are substantially greater than for the observed variation of the stress concentration factor from 3 to 5. Measured improvement factors from two sites, also given on Figure 20, show good agreement with the upper bound equilibrium method curves, equation (22), for n in the range of 3 to slightly less than 5. The curves of Priebe therefore appear, based on a comparison with the equilibrium method and limited field data, to overpredict the beneficial effects of stone columns in reducing settlement.

Greenwood Method. Greenwood [15] has presented preliminary, empirical curves giving the settlement reduction due to ground improvement with stone columns as a function of undrained soil strength and stone column spacing. These curves have been replotted and presented in Fig. 21 using area ratio and improvement factor rather than column spacing and settlement reduction as done in the original curves. In replotting the curves a stone column diameter of 3 ft. (0.9 m) was assumed for the c = 800 psf (40 kN/m^2) upper bound curve and a diameter of 3.5 ft. (1.07 m) for the c = 400 psf (20 kN/m^2) lower bounds curve. Also superimposed on the figure is the equilibrium method upper bounds solution, equation (22) for stress concentration factors of 3, 5, 10 and 20. The Greenwood curve for vibro-replacement and a shear strength of 400 psf (20 kN/m^2) generally corresponds to stress concentration factors of about 3 to 5 for the equilibrium method and hence appears to indicate probable levels of improvement for soft soils for area ratios lessthan about 0.15. For firm soils and usual levels of ground improvement ($0.15 \leq a_s \leq 0.35$), Greenwood's suggested improvement factors indicated on Fig. 21 appear to be high. Stress concentration n decreases as the stiffness of the ground being improved increases relative to the stiffness of the stone column. Therefore, the stress concentration factors greater than 15 required to develop the large level of improvement is unlikely in the firm soil.

In both the Priebe and the Greenwood methods the variables indicated by equation (21) to be of importance in determining the level of improvement are not considered; these effects, however, are generally of secondary importance.

Incremental Method

The method for predicting settlement developed by Goughnour and Bayuk [33] is an important extension of the methods presented earlier by Hughes,

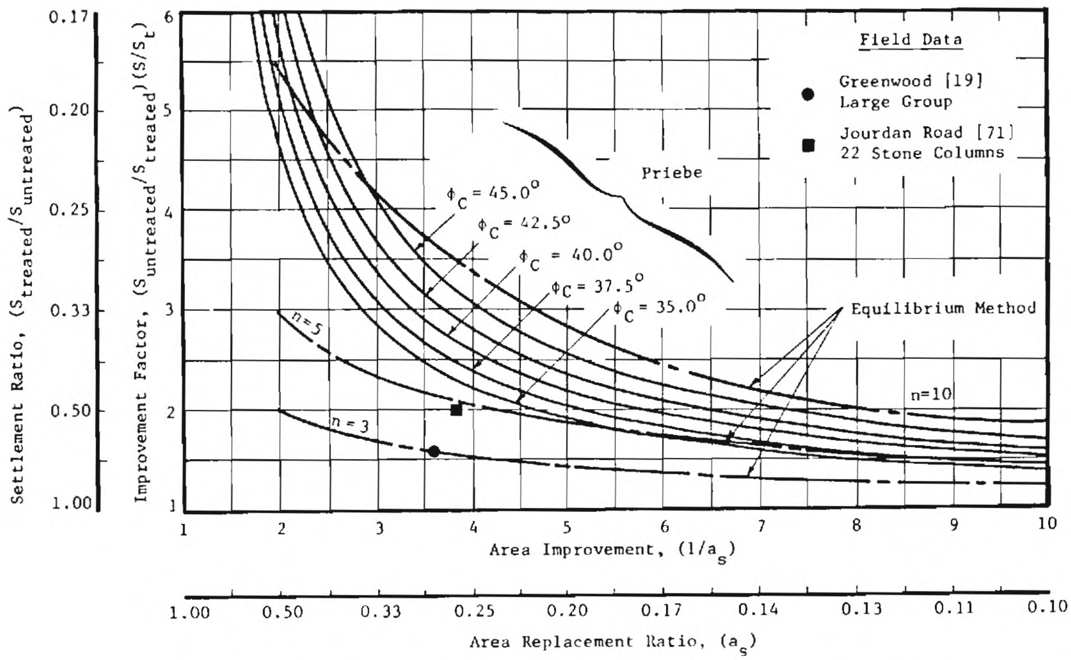


FIGURE 20. SETTLEMENT REDUCTION DUE TO STONE COLUMN - PRIEBE AND EQUILIBRIUM METHODS.

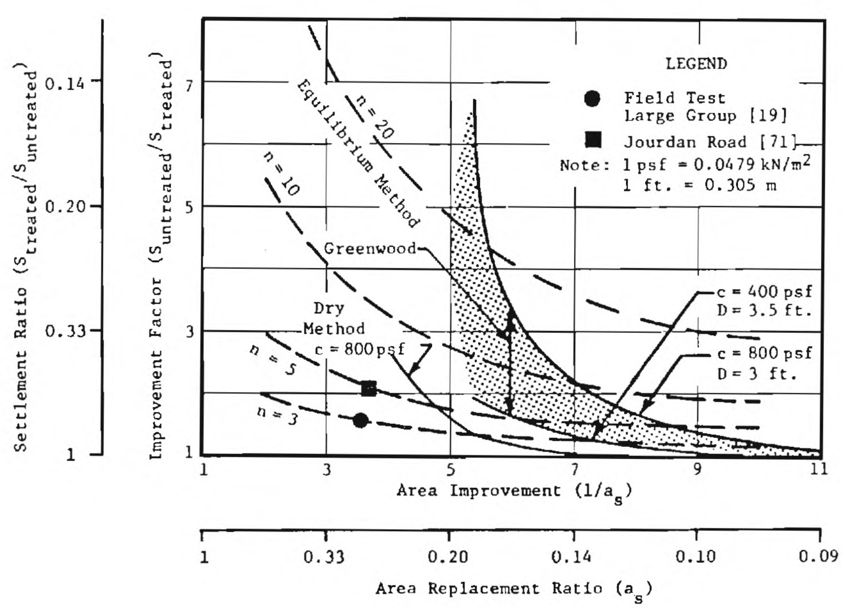


FIGURE 21. COMPARISON OF GREENWOOD AND EQUILIBRIUM METHODS FOR PREDICTING SETTLEMENT OF STONE COLUMN REINFORCED SOIL.

et al. [29], Baumann and Bauer [14], and Priebe [14]. To solve this complicated problem, the unit cell model is used, together with an incremental, iterative, elastic-plastic solution. Although settlements and stresses can be evaluated by hand calculation, a computer solution is necessary from a practical standpoint. Such a computer solution is available, with some restrictions, from the Vibroflotation Foundation Co. [76]. The incremental and the finite element methods are the only ones which give the complete response of the stone column reinforced ground.

Development. The loading is assumed to be applied over a wide area so that the unit cell model can be used in developing the theory. The stone is assumed to be incompressible so that all volume change occurs in the clay. Both vertical and radial consolidation, at least approximately, are considered in the analysis. The unit cell is divided into small, horizontal increments. The vertical strain and vertical and radial stresses are calculated for each increment assuming all variables are constant over the increment.

Both elastic and plastic response of the stone are considered using the incremental method of Goughnour and Bayuk. If stress levels are sufficiently low the stone column remains in the elastic range. For most design stress levels, the stone column bulges laterally yielding plastically over at least a portion of its length. Because of the presence of the rigid unit cell boundaries, a contained state of plastic equilibrium of the stone column in general exists.

The assumption is also made that the vertical, radial and tangential stresses at the interface between the stone and soil are principal stresses. Therefore no shear stresses are assumed to act on the vertical boundary between the stone column and soil. Application of this method and also the finite element studies performed as a part of the present investigation indicates shear stresses acting on the stone column boundaries are generally less than about 200 psf (10 kN/m^2). Because of the occurrence of relatively small shear stresses at the interface, the assumption that vertical and radial stresses are principal stresses appears acceptable as an engineering approximation.

In the elastic range the vertical strain is taken as the increment of vertical stress divided by the modulus of elasticity. The apparent stiffness of the material in the unit cell should be equal to or greater than that predicted by dividing the vertical stress by the modulus of elasticity since some degree of constraint is provided by the boundaries of the unit cell. The vertical strain calculated by this method therefore tends to be an upper (conservative) bound in the elastic range.

Upon failure of the stone within an increment; the usual assumption [11,14,24] is made that the vertical stress in the stone equals the radial stress in the clay at the interface times the coefficient of passive pressure of the stone. Radial stress in the cohesive soil is calculated following the plastic theory developed by Kirkpatrick, Whitman, et al., and Wu, et al. considering equilibrium within the clay [33]. This plastic theory gives the change in radial stress in the clay as a function of the change in vertical stress in the clay, the coefficient of lateral stress in the clay applicable for the stress increment, the geometry, and the initial stress state in the clay. In solving the problem the assumption is made that when the stone column is in a state of plastic equilibrium the clay is also in a plastic state.

Radial consolidation of the clay is considered using a modification of the Terzaghi one-dimensional consolidation theory. Following this approach the Terzaghi one-dimensional equations are still utilized, but the vertical stress in the clay is increased to reflect greater volume change due to radial consolidation. For typical lateral earth pressure coefficients, this vertical stress increase is generally less than about 25 percent, the stress increasing with an increase in the coefficient of lateral stress applicable for the increment in stress under consideration.

Evaluation. The assumptions made in the Incremental Method theoretically are not as sound as those made in the finite element method which will be subsequently discussed. Nevertheless, the theoretical development is felt to simulate reasonably well the stone column construction and loading conditions. Also, the assumptions tend to give an upper bound answer for settlement predictions. From a practical standpoint the input data required to perform a computer analysis are quite simple and include the pertinent material and geometric parameters.

Goughnour and Bayuk [27] obtained encouraging results when compared with settlement measurements from the Hampton, Virginia load test study. Additionally, comparisons were made of the Goughnour-Bayuk method with elastic finite element and equilibrium methods. For a realistic range of stress levels and other conditions the Incremental Method was found to give realistic results which generally fell between the extremes of these two methods. Based on these findings this approach appears to be a viable alternative for estimating settlement of stone column reinforced ground.

Finite Element Method

The finite element method offers the most theoretically sound approach for modeling stone column improved ground. Nonlinear material properties, interface slip and suitable boundary conditions can all be realistically modeled using the finite element technique. Although three-dimensional modeling can be used, from a practical standpoint either an axisymmetric or plane strain model is generally employed. Most studies have utilized the axisymmetric unit cell model to analyze the conditions of either a uniform load on a large group of stone columns [39,40,57] or a single stone column [48,77]; Aboshi, et al. [24] have studied a plane strain loading condition.

An indepth study has been made of stone column behavior using the finite element method by Balaam, Poulos and their co-workers [39,40,57,77]. Balaam, Brown and Poulos [39] analyzed by finite elements large groups of stone columns using the unit cell concept. Undrained settlements were found to be small and neglected. The ratio of modulus of the stone to that of the clay was assumed to vary from 10 to 40, and the Poisson's ratio of each material was assumed to be 0.3. A coefficient of at-rest earth pressure $K_o = 1$ was used. Only about 6 percent difference in settlement was found between elastic and elastic-plastic response. The amount of stone column penetration into the soft layer and the diameter of the column were found to have a significant effect on settlement (Fig. 22); the modular ratio of stone column to soil was of less importance.

Balaam and Poulos [77] found for a single pile that slip at the interface increases settlement and decreases the utlimate load of a single pile which agrees with the findings of Jones and Brown [48]. Also, assuming adhesion at the interface equal to the cohesion of the soil gave good results when compared to field measurements.

Balaam and Booker [78] found, for the unit cell model using linear elastic theory for a rigid loading (equal vertical strain assumption), that vertical stresses were almost uniform on horizontal planes in the stone column and also uniform in the cohesive soil. Also, the stress state in the unit cell was essentially triaxial. Whether the underlying firm layer was rough or smooth made little difference. Based upon these findings, a simplified, linear elasticity theory was developed and design curves were given for predicting performance. Their analysis indicates that as drainage occurs, the vertical stress in the clay decreases and the stress in the stone increases as the clay goes from the undrained to the drained state. This change is caused by a decrease with drainage of both the modulus and Poisson's ratio of the soil.

Development of Design Curves

A finite element study was undertaken to extend this early work and develop design charts for predicting primary consolidation settlement. The finite element program used in this study can solve small or large displacement, axisymmetric or plane strain problems and has been described in detail elsewhere [79,80]. For a nonlinear analysis load was applied in small increments, and computations of incremental and total stresses were performed by solving a system of linear, incremental equilibrium equations for the system. Eight node isoparametric material elements were used in the formulation. Because of the relatively uniform stress condition in the stone and soil, only one vertical column of elements was used to model the stone and one to model the soil.

In selected nonlinear runs interface elements, capable of modeling conditions of no slip, slip, or separation using Mohr-Coulomb failure criteria, were included to define the maximum allowable shear at the interface. At working loads slip was found to only slightly increase settlement, and hence, its effect was not included in developing the nonlinear design curves.

The stiffness of the system was varied after each load increment and iteration. This required extra computer time to form the stiffness matrix for each load increment and iteration, but reduced considerably the number

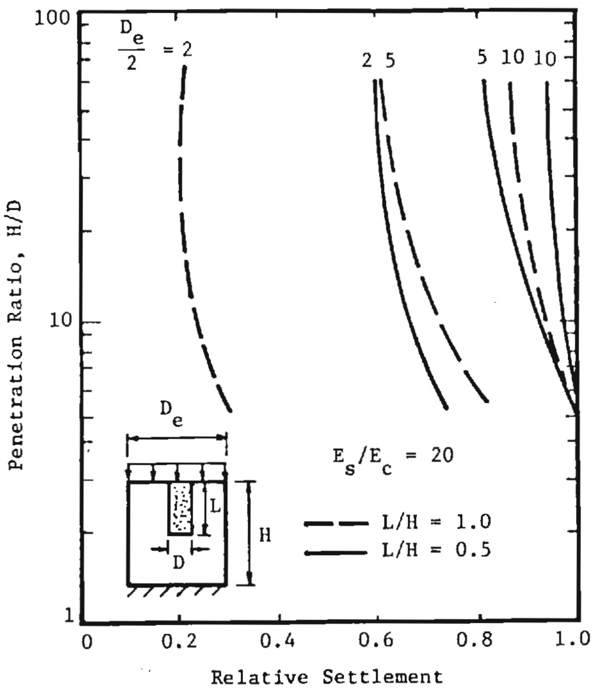
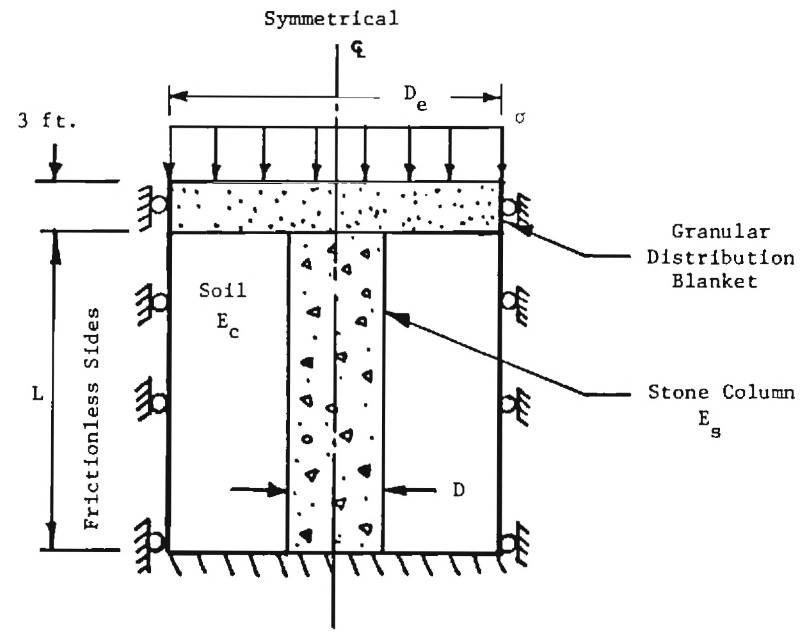


FIGURE 22. EFFECT OF STONE COLUMN PENETRATION LENGTH ON ELASTIC SETTLEMENT [39].



(Note: 1 ft. = 0.305 m)

Definitions: $a_s = A_s/A$ where A_s = area of stone and A = total area

Vertical Settlement, $S = I_s(\frac{P}{E_s L})$ where $P = \sigma \cdot A$

FIGURE 23. NOTATION USED IN UNIT CELL LINEAR ELASTIC SOLUTIONS GIVEN IN FIGS. 24 THROUGH 26.

of iterations for convergence in the nonlinear analysis.

Low Compressibility Soils. Curves for predicting settlement of low compressibility soils such as stone column reinforced sands, silty sands and some silts were developed using linear elastic theory. Low compressibility soils are defined as those soils having modular ratios $E_s/E_c \leq 10$ where E_s and E_c are the average modulus of elasticity of the stone column and soil, respectively. The notation and unit cell model used in the analysis are shown in Fig. 23. The settlement curves for area ratios of 0.1, 0.15 and 0.25 are given in Fig. 24 through 26, respectively. On each figure, curves are given for length to diameter ratios L/D of 5, 10, 15, and 20. The Poisson's ratio of the soil is taken to be 0.30 and of the stone, 0.35.

The elastic finite element study utilizing the unit cell model shows a very nearly linear increase in stress concentration in the stone column with increasing modular ratio (Fig. 27). The approximate linear relation exists for area replacement ratios a_s between 0.1 and 0.25, and length to diameter ratios varying from 4 to 20. For a modular ratio E_s/E_c of 10, a stress concentration factor n of 3 exists (Fig. 27). For soft cohesive soils reinforced with stone columns, the modular ratio can be considerably greater than the upper limit of 40 indicated by Balaam [57] and Balaam and Poulos [77]. For modular ratios greater than about 10, elastic theory underestimates drained settlements primarily due to (1) excessively high stress concentration that theory predicts to occur in the stone and (2) lateral spreading in soft soils. For large stress concentrations essentially all of the stress according to elastic theory is carried by the stone column. Since the stone column is relatively stiff, small settlements are calcaluted using elastic theory when using excessively high stress concentrations.

Compressible Cohesive Soils. Compressible, soft to firm clays, such as encountered at Hampton [27], Clark Fork [10], and Jourdan Road Terminal [71] are prime candidates for reinforcing with stone columns for embankment support. This study and also the work of Datye, et al. [73] indicate for such soft soils the modular ratio between the stone and soil is likely to be in the range of 40 to 100 or more.

To calculate the consolidation settlement in compressible soils (i.e., $E_s/E_c \gtrsim 10$), design curves were developed assuming the clay to be elastic-plastic and the properties of the stone to be stress dependent. The nonlinear stress dependent stiffness characteristics of the stone used in the development of the charts were for the partially crushed gravel used at Santa Barbara [81]. Since a crushed stone is usually used for stone column construction, the stiffness of the Santa Barbara gravel gives a realistic model, slightly on the conservative side. The nonlinear stress-strain properties were obtained from the results of 12 in. (305 mm) diameter triaxial cell test results [81].

In soft clays not reinforced with stone columns, lateral bulging can increase the amount of vertical settlement beneath the fill by as much as 50 percent [82]. In the theoretical model, lateral bulging also reduces

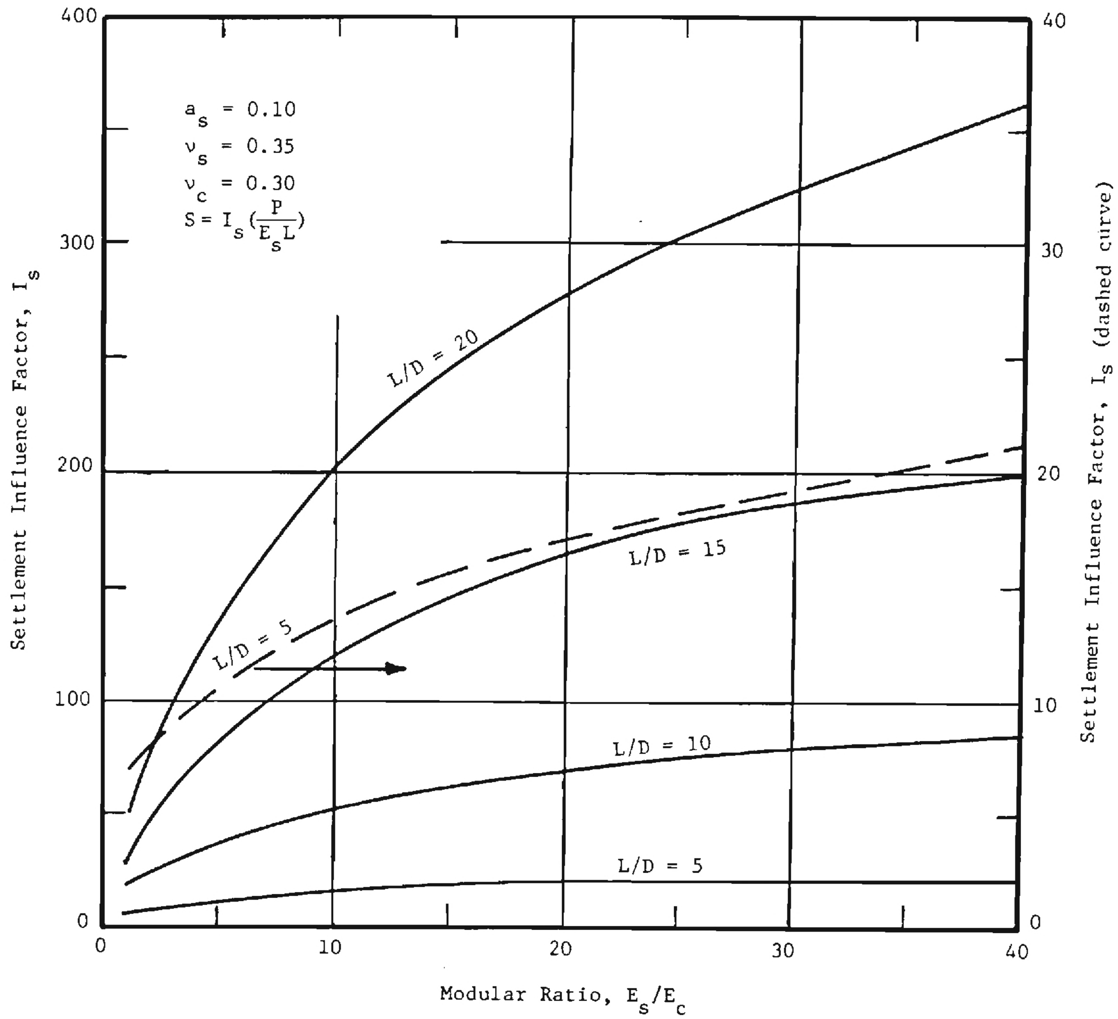


FIGURE 24. LINEAR ELASTIC SETTLEMENT INFLUENCE FACTORS FOR AREA RATIO $a_s = 0.10$ - UNIT CELL MODEL.

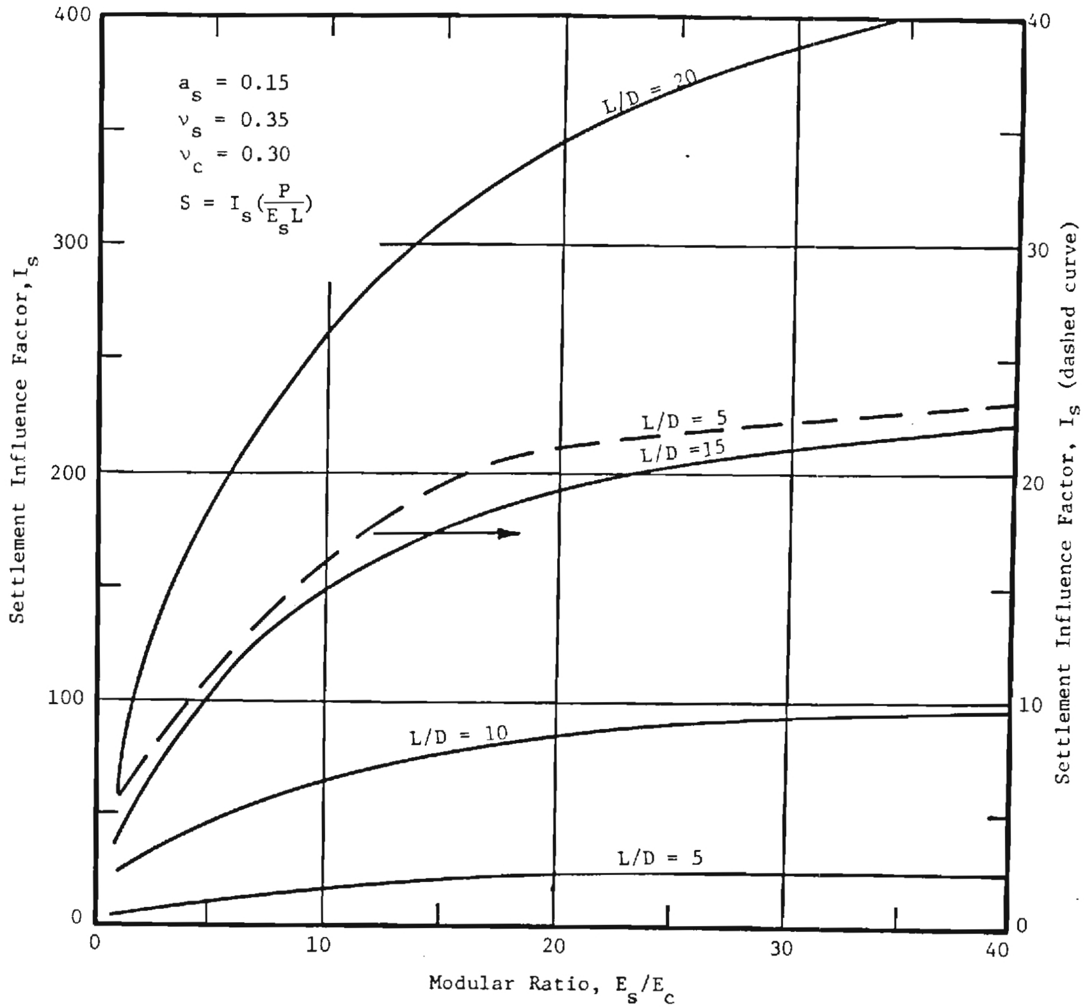


FIGURE 25. LINEAR ELASTIC SETTLEMENT INFLUENCE FACTORS FOR AREA RATIO $a_s = 0.15$ - UNIT CELL MODEL.

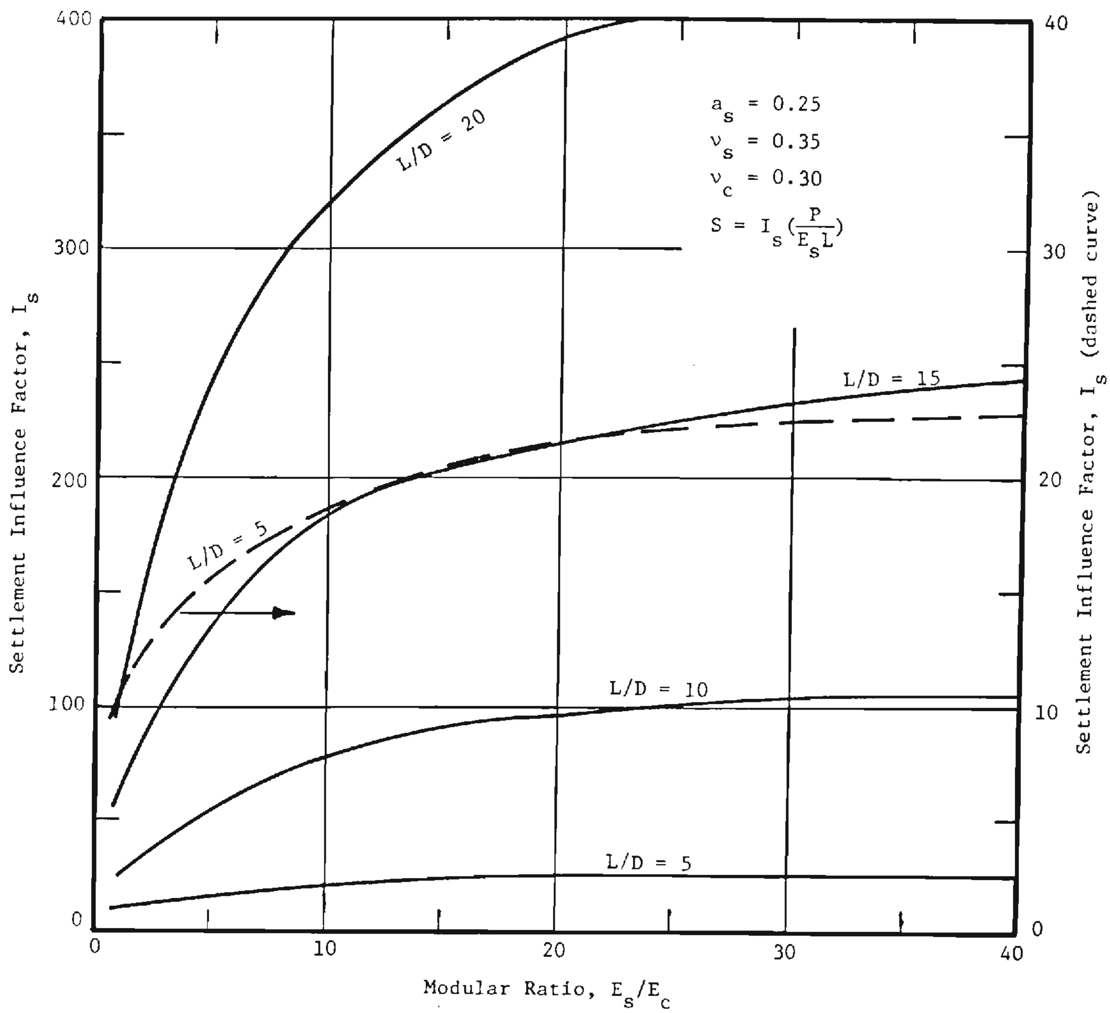


FIGURE 26. SETTLEMENT INFLUENCE FACTORS FOR AREA RATIO $a_s = 0.25$ - UNIT CELL MODEL.

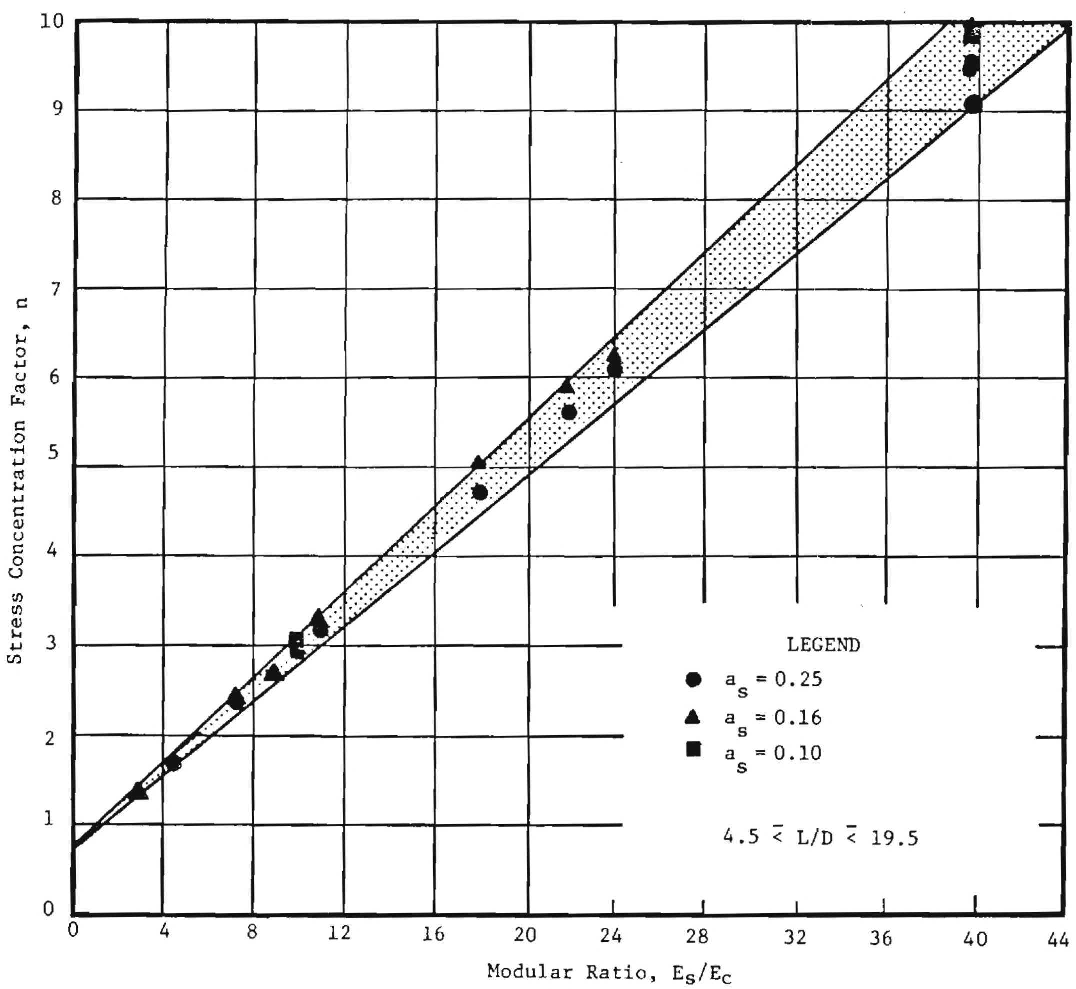


FIGURE 27. VARIATION OF STRESS CONCENTRATION FACTOR WITH MODULAR RATIO - LINEAR ELASTIC ANALYSIS.

the lateral support contributed by the sides of the unit cell. To approximately simulate lateral bulging effects, a soft boundary was placed around the unit cell to allow lateral deformation. Based on the measurements of lateral deformation at Jourdan Road Terminal (Fig. 10) a conservative (maximum) lateral deformation gradient appears to be 0.01 ft./ft. This gradient represents the amount of lateral deformation that might occur over a horizontal distance of one unit. From this deformation gradient, the maximum amount of bulging that would be likely to occur across the unit cell was estimated. By trial and error using the finite element analysis, a boundary 1 in. (25 mm) thick having an elastic modulus of 12 psi (83 kN/m^2) was found to model the maximum lateral deformations caused by lateral spreading that should occur across the unit cell. To obtain the possible variation in the effect of boundary stiffness (lateral spreading), a relatively rigid boundary was also used, characterized by a modulus of 1,000 psi (6900 kN/m^2) (Fig. 28). The deformation gradient of course is not a constant and would vary with many factors including the stiffness of the soil being reinforced, the applied stress level and the level of ground improvement used. Therefore the above approach should be considered as a first engineering approximation.

The unit cell model and notation used in the analysis is summarized in Fig. 28. The design charts developed using this approach are presented in Figs. 29 through 37. Settlement is given as a function of the uniform, average applied pressure σ over the unit cell, modulus of elasticity of the soil E_c, area replacement ratio a_s, length to diameter ratio, L/D, and boundary rigidity. For design the average modulus of elasticity of compressible cohesive soils can be determined from the results of one-dimensional consolidation tests using equation (47). The charts were developed for a representative angle of internal friction of the stone $\phi_s = 42°$, and a coefficient of at-rest earth pressure K_o of 0.75 for both the stone and soil. For soils having a modulus E_c equal to or less than 160 psi (1100 kN/m^2), the soil was assumed to have a shear strength of 400 psf (19 kN/m^2). Soils having greater stiffness did not undergo an interface or soil failure; therefore, soil shear strength did not affect the settlement.

Fig. 38 shows the theoretical variation of the stress concentration factor n with the modulus of elasticity of the soil and length to diameter ratio, L/D. Stress concentration factors in the range of about 5 to 10 are shown for short to moderate length columns reinforcing very compressible clays (E_c < 200 to 300 psi, 1380-2070 kN/m^2). These results suggest that the nonlinear theory may predict settlements smaller than those observed. A comparison of measured and calculated settlements is presented in Chapter VII.

STRESS DISTRIBUTION IN STONE COLUMN GROUPS

Many stone column applications such as bridge pier and abutment foundations involve the use of stone column groups of limited size. A knowledge of the stress distribution within the stone column improved soil is necessary to estimate the consolidation settlement. Also the vertical stresses in the stone columns and cohesive soil are of interest in performing stability

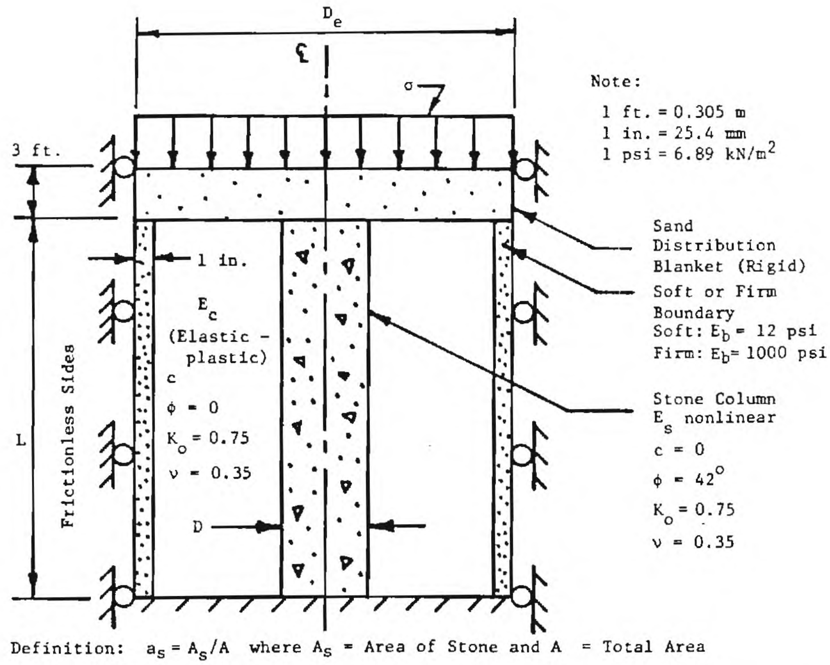


FIGURE 28. NOTATION USED IN UNIT CELL NONLINEAR SOLUTIONS GIVEN IN FIGURES 29 THROUGH 37.

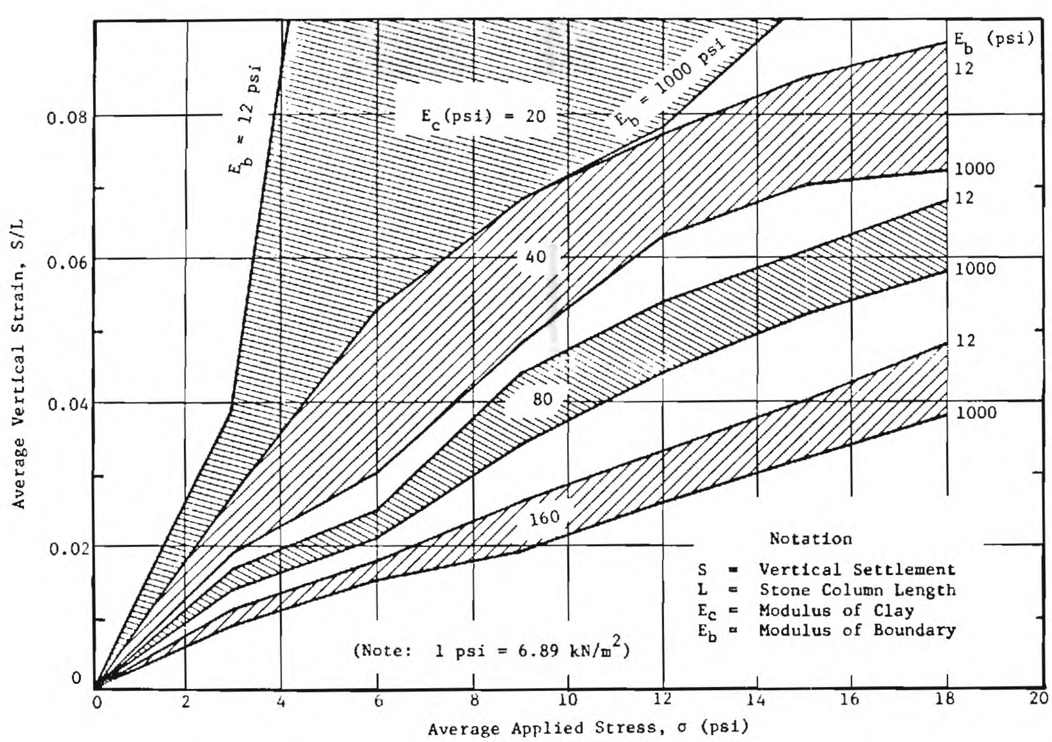


FIGURE 29. NONLINEAR FINITE ELEMENT UNIT CELL SETTLEMENT CURVES: $a_s = 0.10$, L/D = 5.

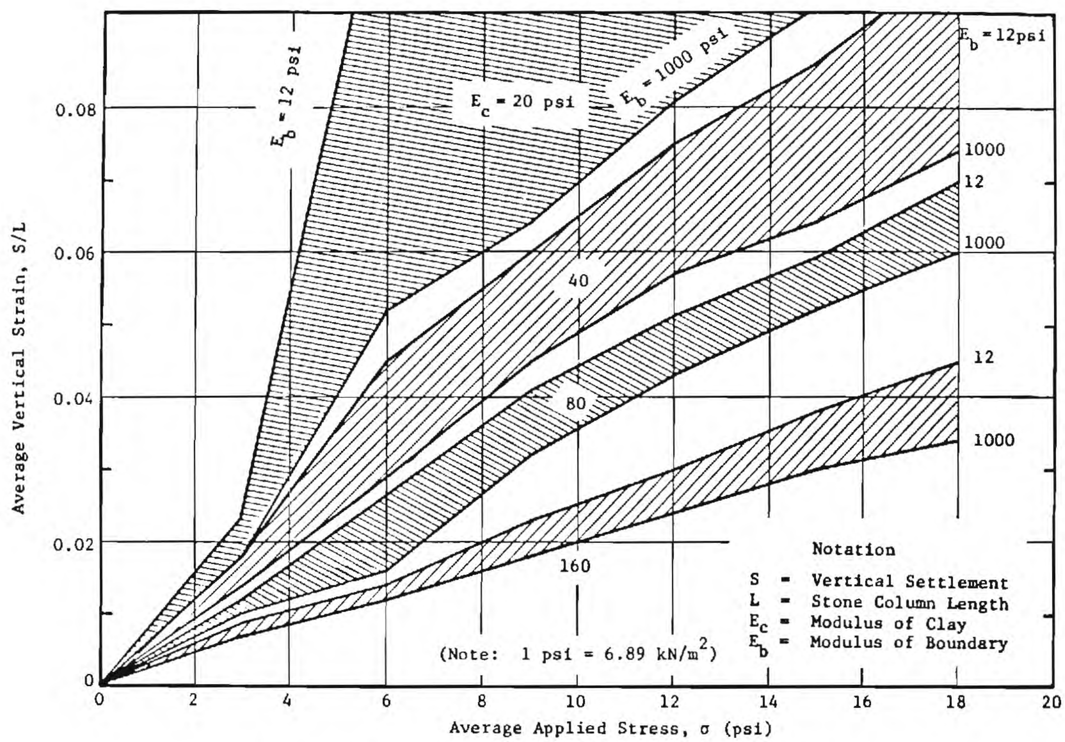


FIGURE 30. NONLINEAR FINITE ELEMENT UNIT CELL SETTLEMENT CURVES: $a_s = 0.10$, $L/D = 10$.

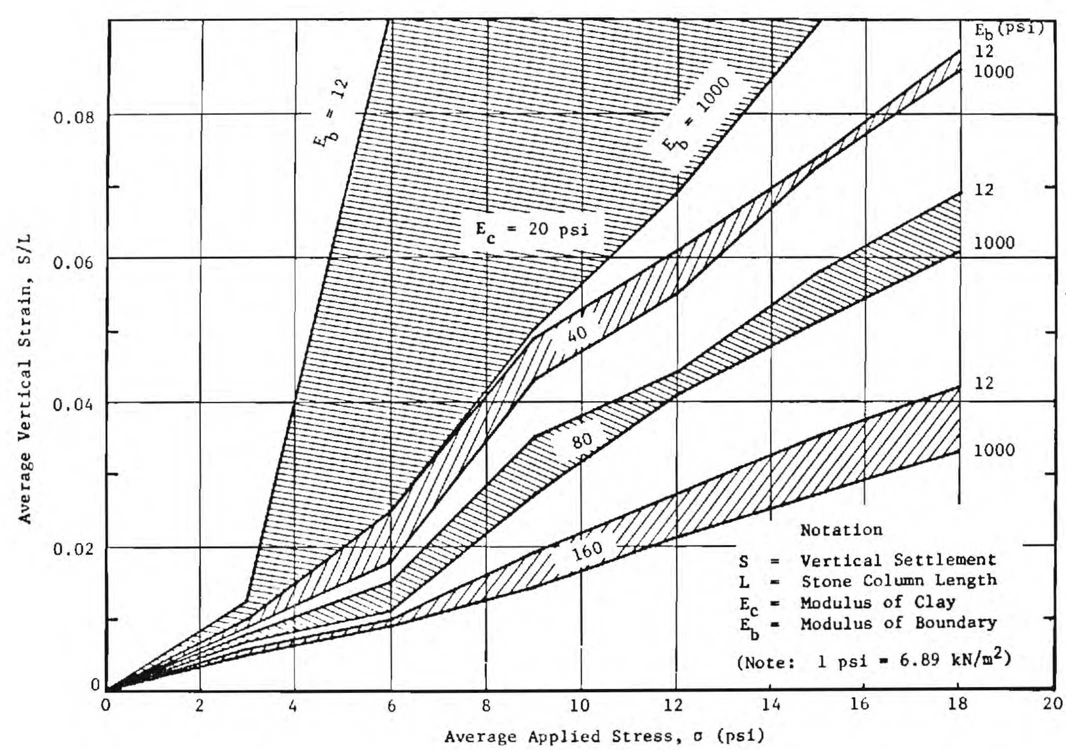


FIGURE 31. NONLINEAR FINITE ELEMENT UNIT CELL SETTLEMENT CURVES: $a_s = 0.10$, $L/D = 20$.

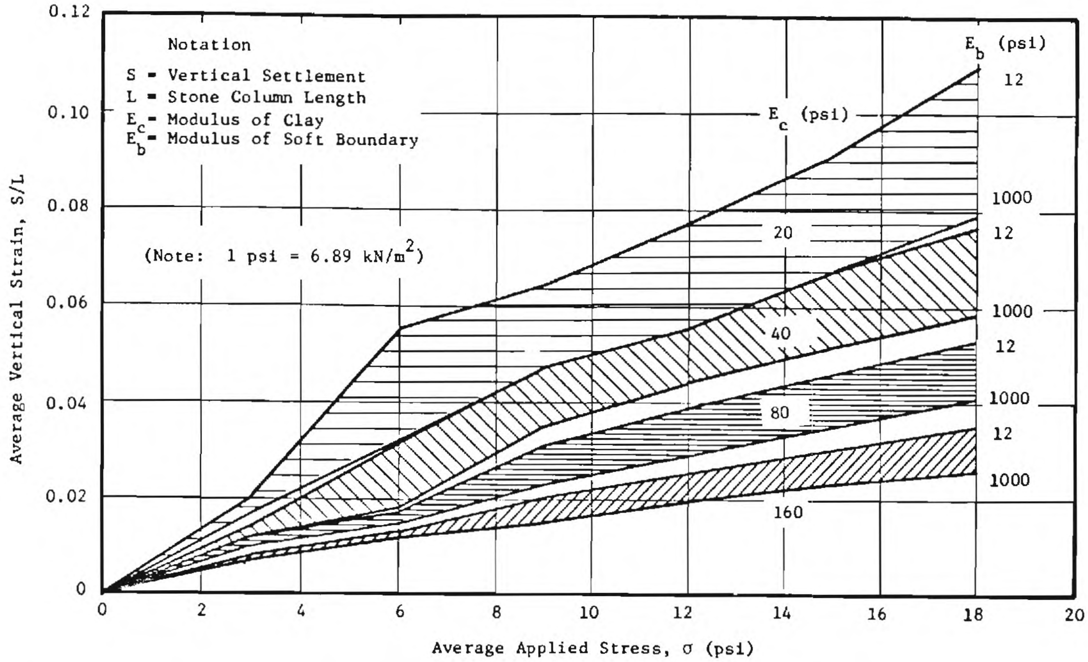


FIGURE 32. NONLINEAR FINITE ELEMENT UNIT CELL SETTLEMENT CURVES: $a_s = 0.25$, L/D = 5.

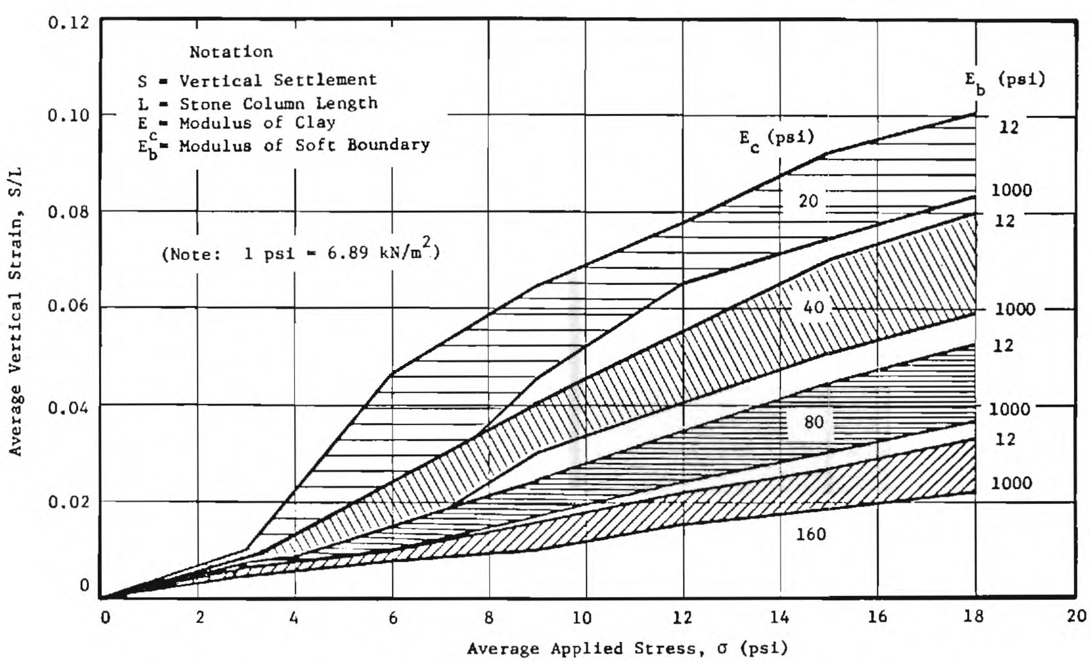


FIGURE 33. NONLINEAR FINITE ELEMENT UNIT CELL SETTLEMENT CURVES: $a_s = 0.25$, L/D = 10.

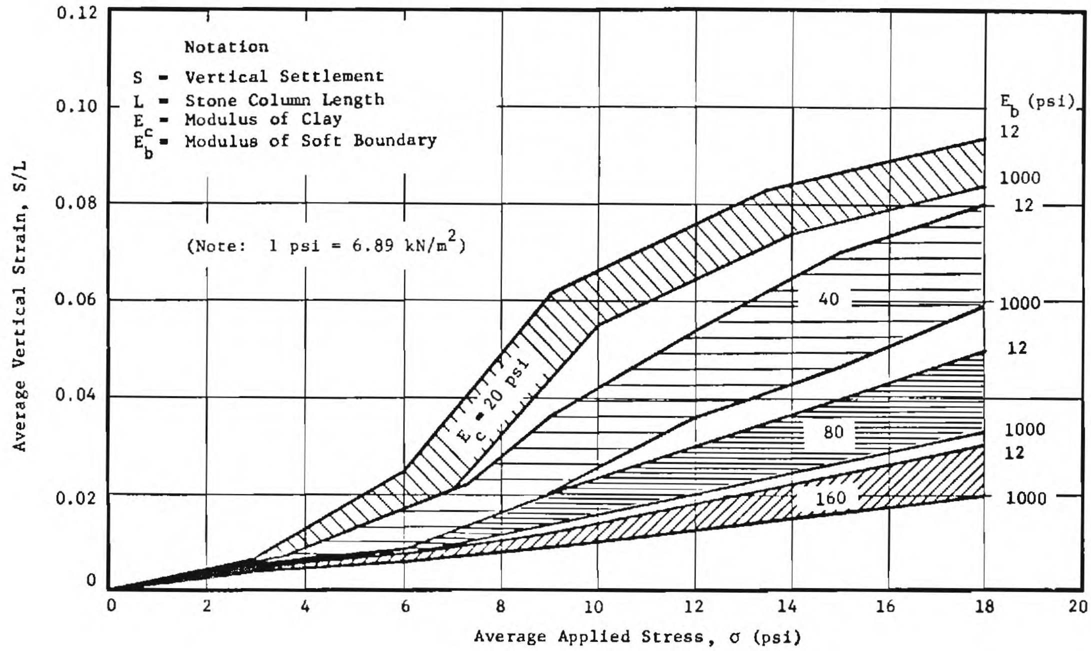


FIGURE 34. NONLINEAR FINITE ELEMENT UNIT CELL SETTLEMENT CURVES: $a_s = 0.25$, L/D = 15.

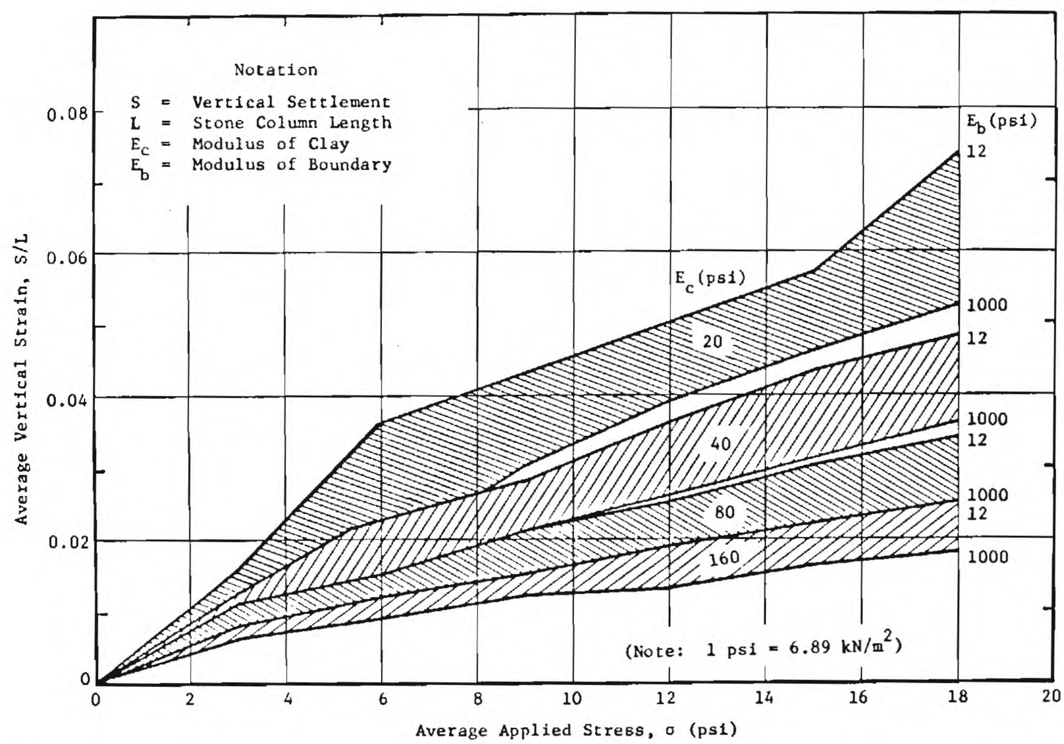


FIGURE 35. NONLINEAR FINITE ELEMENT UNIT CELL SETTLEMENT CURVES: $a_s = 0.35$, L/D = 5.

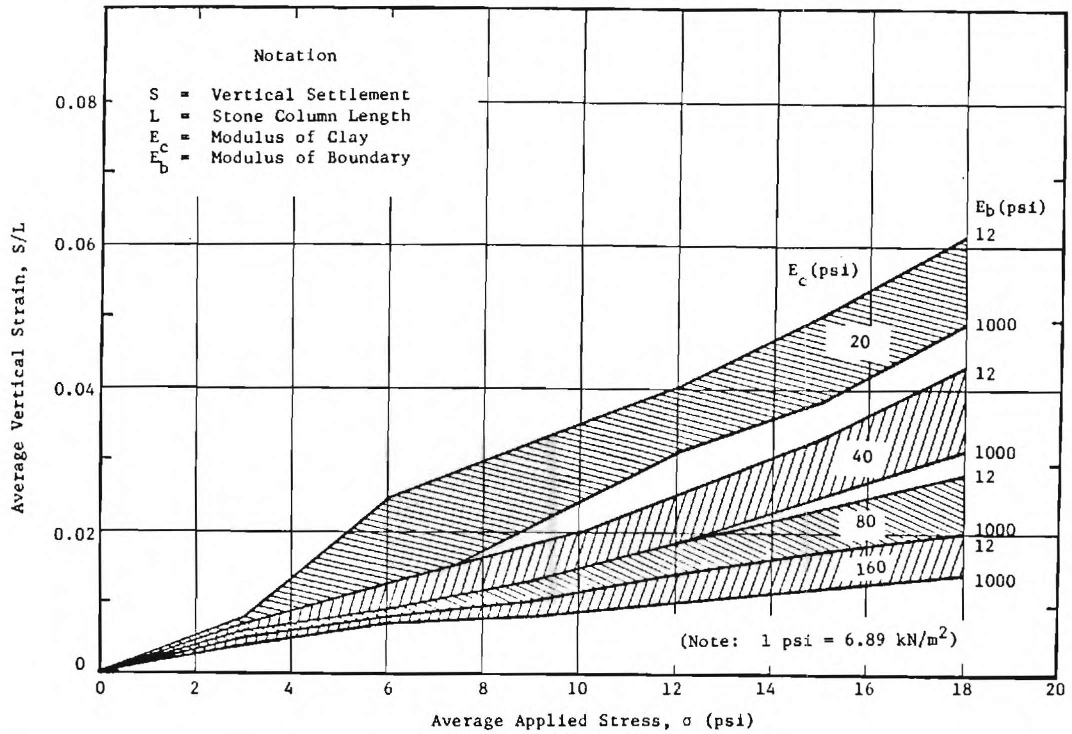


FIGURE 36. NONLINEAR FINITE ELEMENT UNIT CELL SETTLEMENT CURVES: $a_s = 0.35$, L/D = 10.

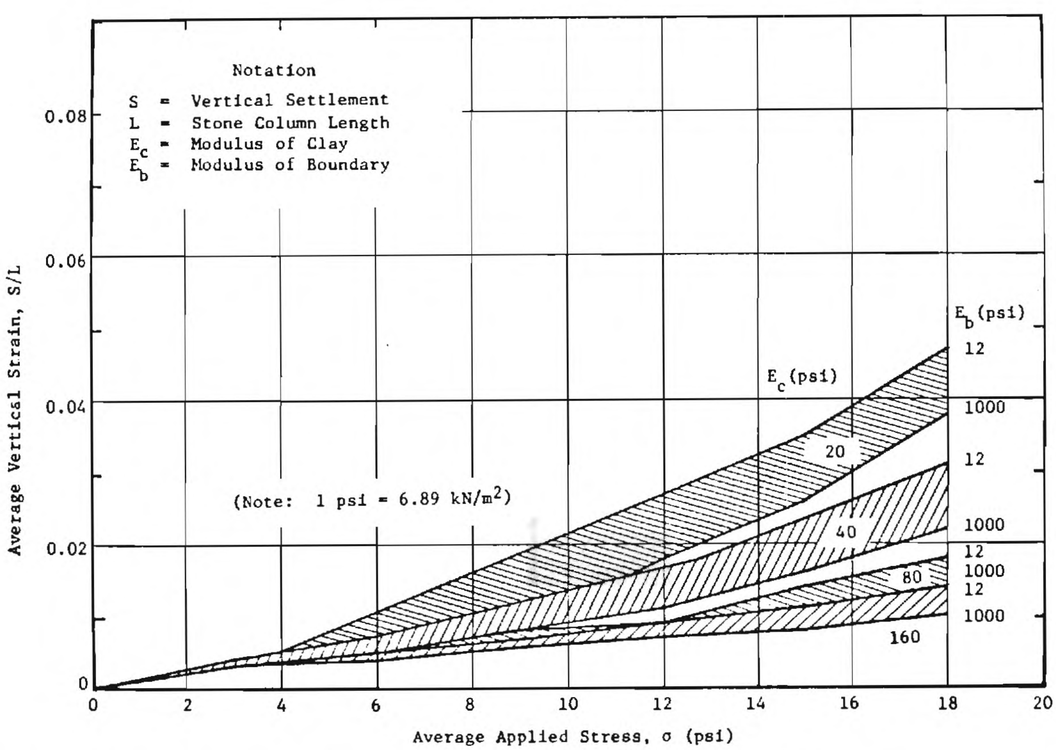


FIGURE 37. NONLINEAR FINITE ELEMENT UNIT CELL SETTLEMENT CURVES: $a_s = 0.35$, L/D = 20.

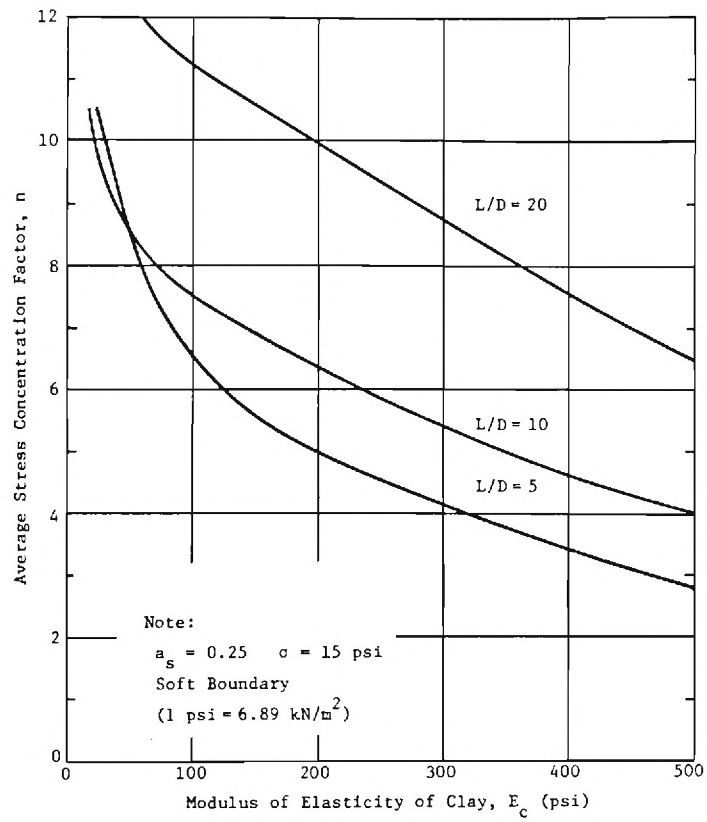


FIGURE 38. VARIATION OF STRESS CONCENTRATION WITH MODULAR RATIO - NONLINEAR ANALYSIS.

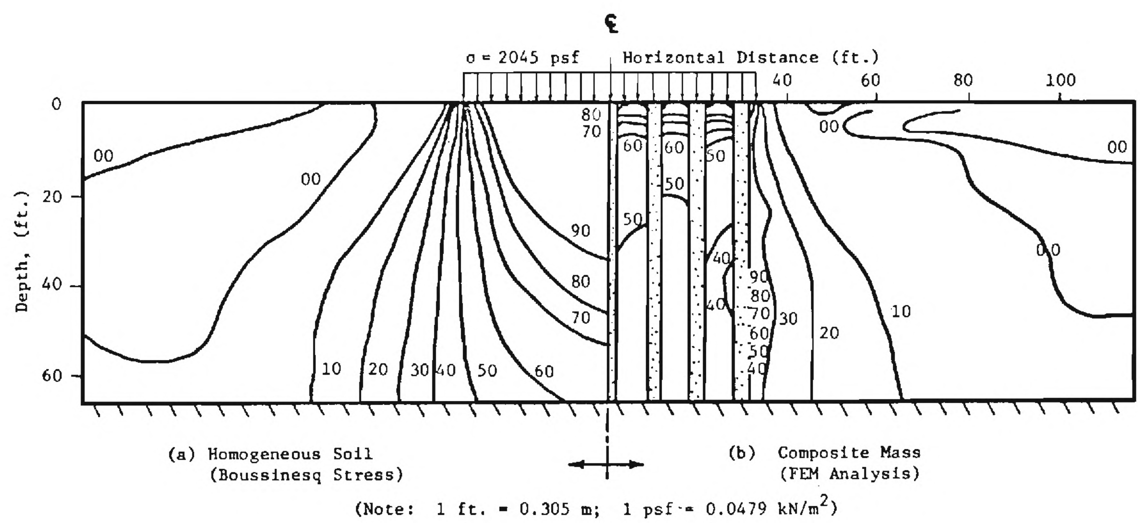


FIGURE 39. COMPARISON OF BOUSSINESQ STRESS DISTRIBUTION WITH A FINITE ELEMENT ANALYSIS OF THE COMPOSITE MASS - PLANE STRAIN LOADING [24].

analyses.

The stress applied to a stone column group of limited size spreads out laterally with depth into the surrounding cohesive soil. The spreading of vertical stress into the soil surrounding the stone columns is similar to that which occurs in a homogeneous soil. In stone column reinforced ground, however, the presence of the relatively stiff columns beneath the foundation would be expected to perhaps concentrate the stress in the vicinity of the stone columns more than in a homogeneous soil. Also, the vertical stress in the stone column is greater than in the adjacent cohesive soil.

Aboshi, et al. [24] have presented results of a finite element study comparing the vertical distribution of stress in ground reinforced with sand compaction piles to a homogeneous soil. The same infinitely long, uniform strip loading was applied to each type soil. In the reinforced ground the stiff columns extended to near the sides of the load, with the width of loading being equal to the depth of the reinforced layer as shown in Fig. 39. This figure shows contours of vertical stress in the reinforced ground on the right side and in the homogeneous soil on the left. The vertical stress in the cohesive soil just outside the edge of the reinforced soil is quite similar to the vertical stress outside the loading in the homogeneous soil.

The best approach at the present time for estimating the vertical stress distribution beneath loadings of limited size supported by stone column reinforced ground is to perform a finite element analysis. A practical approximate approach, however, is to use Boussinesq stress distribution theory as illustrated in Fig. 40. Following this method the average vertical stress σ_i at any desired location within the stone column group is calculated using Boussinesq stress distribution theory and the applied stress σ. Therefore, considering stress concentration effects the vertical stress in the clay can be taken as $\sigma_c = \mu_c \sigma_i$ and in the stone $\sigma_s = \mu_s \sigma_i$ where μ_c and μ_s have been previously defined by equations (8a) and (8b). This approach, although admittedly approximate, is easy to apply in practice and gives realistic estimates of stress distribution and resulting settlements.

RATE OF PRIMARY CONSOLIDATION SETTLEMENT

In a cohesive soil reinforced with stone columns, water moves toward the stone column in a curved path having both vertical and radial components of flow as illustrated in Fig. 41. Newman [83] has shown by the method of separation of variables that this problem can be correctly solved by considering the vertical and radial consolidation effects separately. Following this approach the average degree of primary consolidation of the layer can be expressed as:

$$U = 1-(1-U_z)(1-U_r) \tag{25}$$

where: U = the average degree of consolidation of the coehsive layer considering both vertical and radial drainage

U_z = the degree of consolidation considering only vertical flow

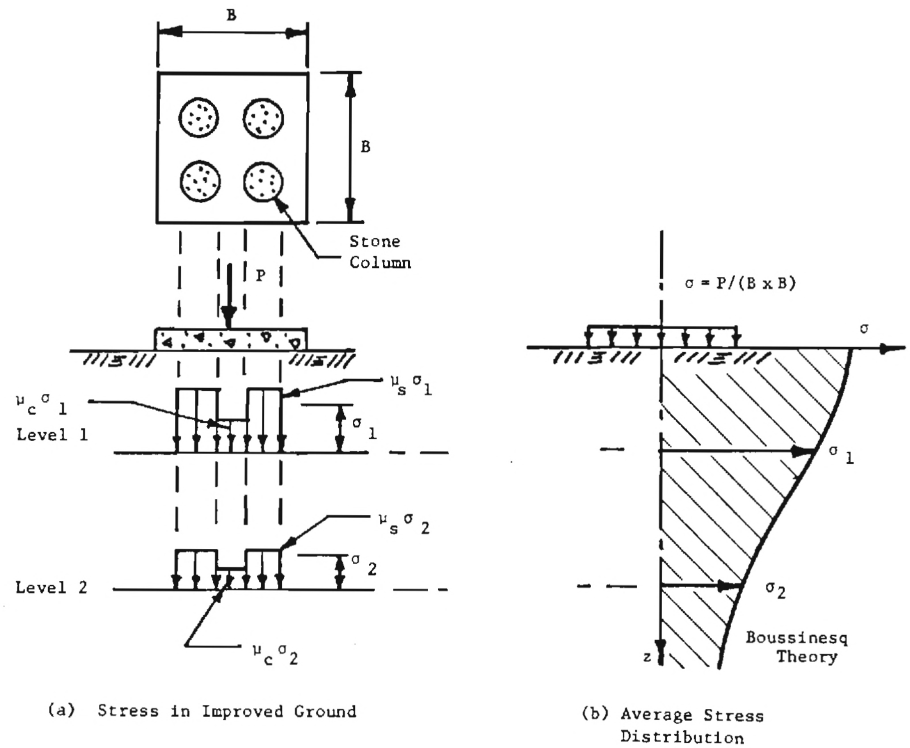


FIGURE 40. APPROXIMATE METHOD OF CALCULATING STRESS DISTRIBUTION IN IMPROVED GROUND SUBJECTED TO LOAD OF FINITE WIDTH.

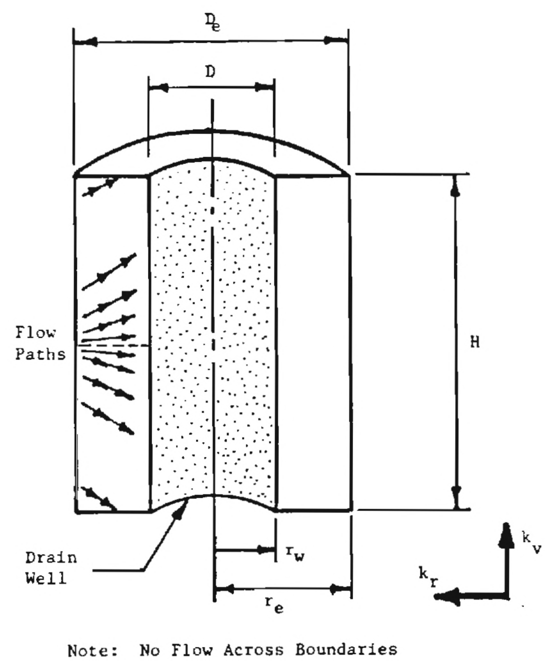


FIGURE 41. FLOW OF WATER WITHIN UNIT CELL TO STONE COLUMN DRAIN - SECTION VIEW [85].

U_r = the degree of consolidation considering only radial flow

In the above expression U, U_z, and U_r are all expressed as a fraction. The primary consolidation settlement at time t of a cohesive layer reinforced with stone columns is:

$$S_t' = U \cdot S_t \tag{26}$$

where: S_t' = primary consolidation settlement at time t

S_t = ultimate primary consolidation settlement of treated ground

U = average degree of consolidation given by equation (25)

Following the Terzaghi one-dimensional consolidation theory, the degree of consolidation in the vertical direction, U_z, is given in Fig. 42 as a function of the dimensionless time factor T_z [74]. The time factor for the vertical direction is expressed as:

$$T_z = C_\nu t/(H/N)^2 \tag{27}$$

where: T_z = time factor for vertical direction

C_ν = coefficient of consolidation in vertical direction

t = elapsed time

H = thickness of cohesive layer

N = number of permeable drainage surfaces at the top and/or bottom of the layer (N = 1 or 2)

The Terzaghi one-dimensional consolidation theory has been extended to include radial flow [84,85]. The degree of consolidation in the radial direction, U_r, as a function of the dimensionless time factor T_r is given in Fig. 43. The time factor for radial drainage is given by:

$$T_r = \frac{C_{\nu_r} t}{(D_e)^2} \tag{28}$$

where: T_r = time factor for radial drainage

C_{ν_r} = coefficient of consolidation in radial direction

t = elapsed time of consolidation

D_e = equivalent diameter of unit cell

The solution given in Fig. 43 for radial consolidation assumes the stone column and the soil to settle equal amounts (i.e., an equal strain assumption). Richart [85] has shown that the equal strain solution and the free strain solution are essentially the same for a degree of consolidation greater than about 50 percent; only modest differences exist between the two solutions for lower degrees of consolidation. Further, Vautrain [63] and the present finite element study have indicated approximately equal settlements to occur in stone column reinforced ground. Therefore, the equal

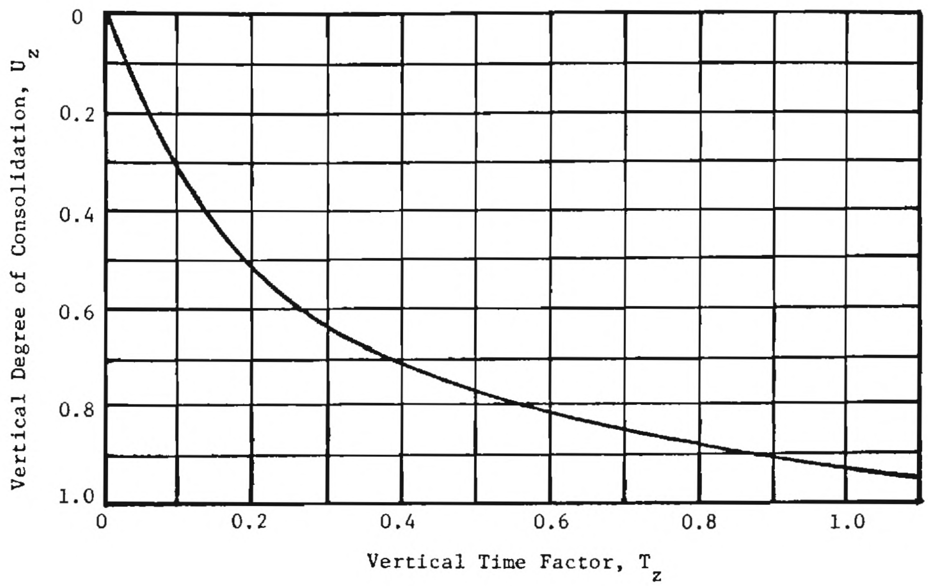


FIGURE 42. DEGREE OF CONSOLIDATION IN VERTICAL DIRECTION.

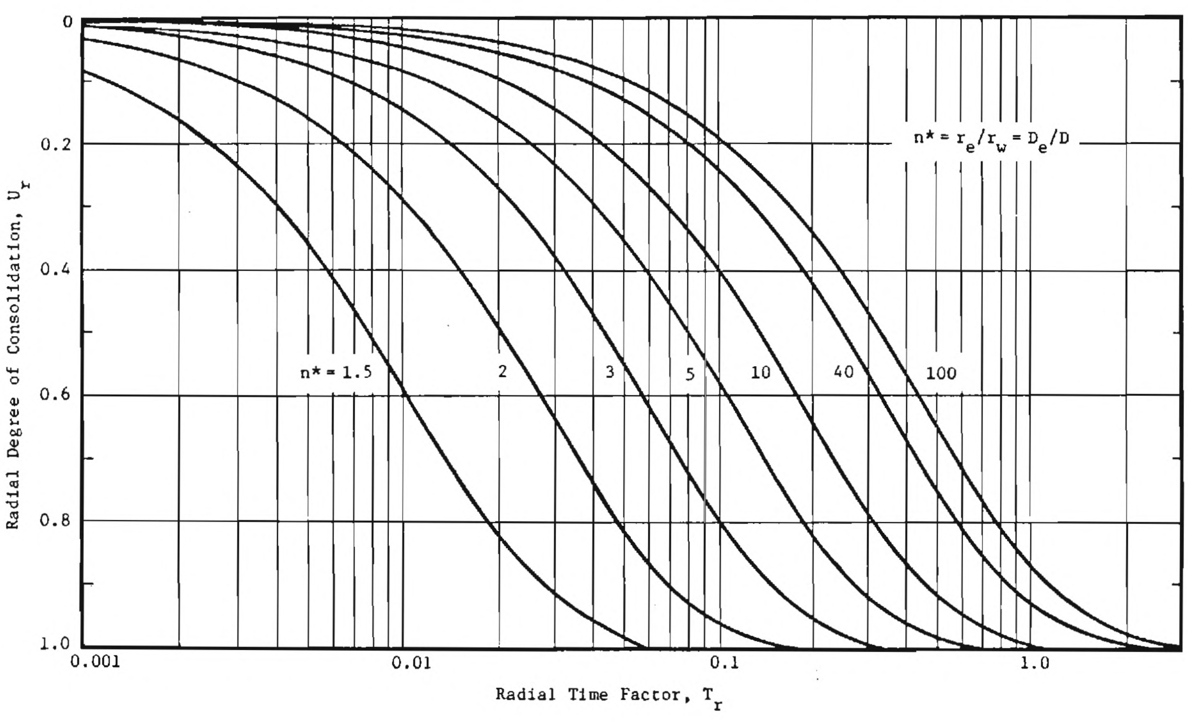


FIGURE 43. DEGREE OF CONSOLIDATION IN RADIAL DIRECTION.

strain assumption is reasonable.

The assumptions made in developing the Terzaghi one-dimensional consolidation theory are as follows [85]:

1. The soil is saturated with an incompressible fluid
2. The mineral components (solids) are incompressible
3. Darcy's Law is valid
4. The coefficient of permeability is a constant
5. The coefficient of compressibility, a_v is a constant for the applied range of pressure
6. The void ratio e is a constant

Additional assumptions made in the derivation for radial drainage include: (7) the drain is infinitely permeable and incompressible, and (8) only vertical compression occurs (i.e., lateral flow of water takes place but no lateral strain). As a result of assumptions 4 through 6, the coefficient of consolidation is assumed in the theory to be a constant. Since the coefficient of consolidation actually varies with stress, it must be selected at stress levels representative of field conditions. The assumption of no lateral deformation is open to question since important lateral movements can occur beneath embankments supported on stone columns. Richart, however, has shown that consolidation is only moderately affected by changes in void ratio from 0.9 to 0.4; also, the assumption of a constant void ratio is conservative.

Smear

In constructing stone columns (or sand drains) a zone of soil adjacent to the column becomes smeared. Further, the soil immediately adjacent to the stone column is disturbed, and soil may intrude into the pores of the stone near the periphery. These factors reduce the permeability of a zone around the outside of the stone column, and hence reduces its effectiveness in draining water radially. The combined effects of smear, disturbance, and intrusion is generally simply referred to as "smear."

The reduction in radial flow due to the presence of smear can be correctly handled mathematically using a hydraulically equivalent system without smear [85]. In the equivalent system without smear, the radius of the drain is reduced the necessary amount to give the same radial flow as occurs in the system with smear. To determine the equivalent system let

s^* = radius of smear zone (r_s) divided by the radius of the active drain (r_w), $s^* = r_s/r_w$

n^* = radius of the unit cell (r_e) divided by the radius of the drain (r_w), $n^* = r_e/r_w$

n^*_{equiv} = radius of unit cell (r_e) divided by the equivalent radius of the drain without smear (r^*_w), $n^*_{equiv} = r_e/r^*_w$

In this discussion k_r is the radial soil permeability and k_s is the radial permeability of the smear zone. Now assume s^*, n^* k_r and k_s are all known or can be estimated. Fig. 44 can then be used to determine n^*_{equiv}. The reduced drain radius for the system without smear hydraulically equivalent to the system with smear is then equal to

$$r^*_w = r_e/n^*_{equiv} \tag{29a}$$

The terms used in the above expressions are illustrated in Fig. 45.

Sometimes a value of the smear factor S^* is assumed to indicate the amount of smear around the column. The smear factor S^* is expressed as follows:

$$S^* = k_r(s^*-1)/k_s \tag{29b}$$

where all terms have been previously defined. In this case either s^* or k_r/k_s is assumed and the other unknown term calculated using equation (29b). Fig. 44 can be used to determine the hydraulically equivalent radius as previously described.

Summary

The theoretical procedures presented for determining the time rate of primary consolidation settlement in stone column reinforced cohesive soils are based on the assumptions made for the Terzaghi one-dimensional consolidation theory. Based on a comprehensive study of sand drains, Rutledge and Johnson [87] have concluded that the consolidation theory is valid. In applications, however, limited accuracy is frequently obtained because of the practical problem of determining representative physical properties for use in the theory. However, time-rate of settlement estimates are usually on the conservative side.

SECONDARY COMPRESSION SETTLEMENT

As water is slowly squeezed from the pores of a cohesive soil due to the applied loading, the effective stress increases and primary consolidation occurs. After the excess pore pressures caused by the loading have dissipated, a decrease in volume of the cohesive soil resulting in settlement continues to occur under constant effective stress [88]. This type volume change occurring under a constant effective stress is called secondary compression (or secondary consolidation). Secondary compression actually starts during the primary consolidation phase of settlement.

Theory

The theory for estimating secondary compression is based on the observation that the relationship between secondary settlement and the logarithm of time can often be approximated by a straight line. Now consider the

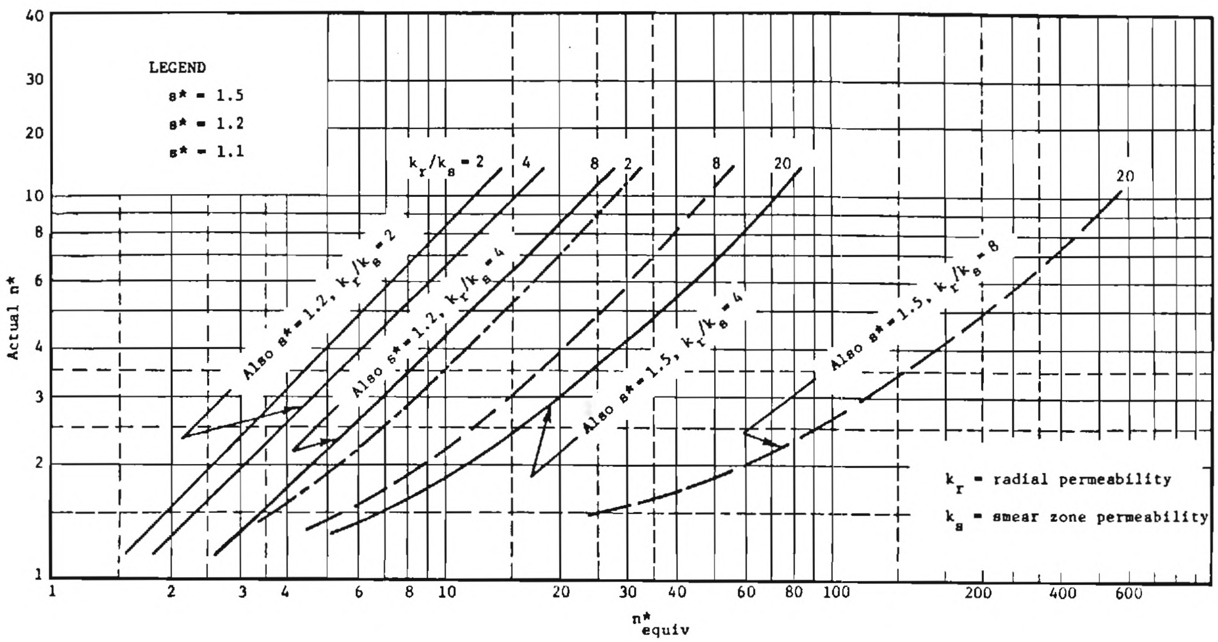


FIGURE 44. ESTIMATION OF n^*_{equiv} FOR STONE COLUMN DRAIN WITHOUT SMEAR.

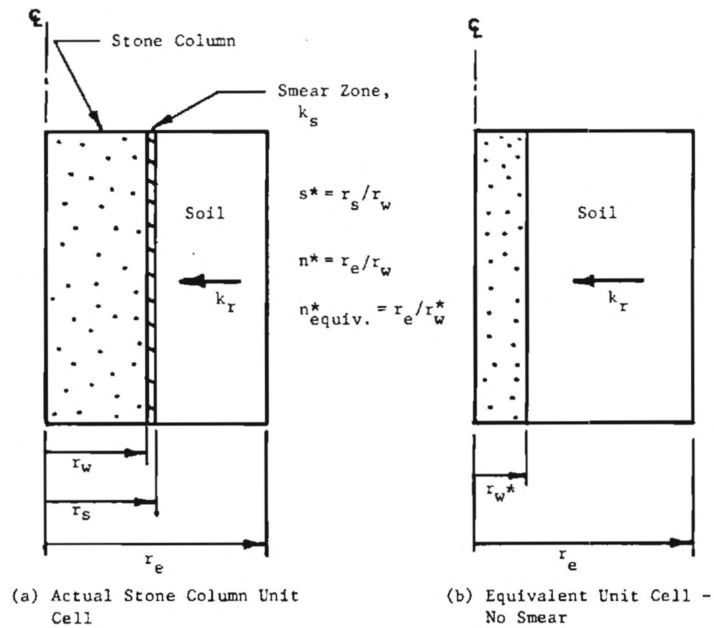


FIGURE 45. EQUIVALENT STONE COLUMN UNIT CELLS WITH AND WITHOUT SMEAR ZONE [85].

amount of secondary compression occurring in a selected layer (or sublayer) of thickness H subjected to an average vertical stress increase of σ_c. Assume a straight line relationship to exist between secondary compression and the logarithm of time. Secondary settlement can then be calculated from the equation of the straight line using:

$$\Delta S = C_\alpha H \log_{10} \frac{t_2}{t_1} \tag{30}$$

where: ΔS = secondary compression of the layer

C_α = a physical constant evaluated by continuing a one-dimensional consolidation test past the end of primary consolidation for a suitable load increment

H = thickness of compressible layer

t_1 = time at the beginning of secondary compression; the time corresponding to 90 percent of primary consolidation is sometimes used

t_2 = time at which the value of secondary compression is desired

The results of one-dimensional consolidation tests are used to evaluate the constant C_α in equation (30). The load increments in the consolidation test are left on sufficiently long to establish for secondary settlement the relationship between the dial reading and logarithm of time. The constant C_α is evaluated from the consolidation test plot of dial reading versus logarithm of time by solving equation (30). The constant C_α should of course be evaluated for an initial stress and stress change which approximately corresponds to the average stress conditions occurring in the field in the layer under consideration. Usually C_α increases with increasing stress level. For stone column improved ground, change in stress in the cohesive soil due to the applied loading can be estimated using equation (8a).

In applying equation (30), secondary compression occurring before time t_1 is neglected. Secondary compression calculated by equation (30) assumes one-dimensional compression if the conventional consolidation test is used to evaluate C_α; hence the unit cell concept is assumed valid. Some radial consolidation would occur even in the unit cell as stone is forced radially into the soil.

Finally, Leonards [88] has pointed out that secondary settlement does not always vary linearly with the logarithm of time. As a result of these and other complicating factors, secondary compression is even harder to predict than one-dimensional consolidation settlement. Therefore settlement predicted from the above approach should be considered as a crude estimate.

SLOPE STABILITY ANALYSIS

Introduction

General stability of the earth mass is often a serious problem when embankments are constructed over soft underlying soils. Use of stone columns to improve the underlying soft soil is one viable alternative for increasing to an acceptable level the safety factor with respect to a general rotational or linear type stability failure. Stone columns are also used to increase the stability of existing slopes under-going landslide problems. A stability analysis of an embankment or landslide stabilized using stone columns is performed mechanistically in exactly the same manner as for a normal slope stability problem except stress concentration must be considered. The Simplified Bishop Method of Slices [9,89,90] is recommended for analyzing stability problems for soil conditions in which a circular rotational failure would be expected to occur. The method of stability analysis can also be used as an approximation to evaluate stability when heavy loads over large areas (such as oil and water tanks) are applied to stone column reinforced ground [24,66].

A computer slope stability analysis when possible should be used to permit considering more trial circles and design conditions, and to minimize errors. A review of the use and limitations of computer slope stability programs has been given elsewhere [89,90]. Unfortunately readily available computer programs such as LEASE I [91], LEASE II [122] and STABL [123] were not specifically developed to handle the problem of stress concentrations in stone column reinforced ground. Therefore, although these and other computer programs can be used for stability problems involving stone columns, limitations exist on their use and some adaption of the input data and/or computer program is required. Three general techniques that can be used to analyze the stability of stone column reinforced ground are described in this section.

Profile Method

The profile method can be used for computer analysis of stone column reinforced ground using a slope stability program having the general capabilities of LEASE [91,122]. In the profile method each row of stone columns is converted into an equivalent, continuous stone column strip with width w. The continuous strips have the same volume of stone as the tributary stone columns as shown in Fig. 46. Each strip of stone and soil is then analyzed using its actual geometry and material properties. In a computer analysis each individual stone column strip and soil strip is input together with their respective properties. Data input to the computer, which is tedious using this method, could be somewhat reduced by developing an automatic data generating computer routine.

Stress Concentration. In landslide problems stress concentration for many applications would not develop. However, when stone columns are used to improve a soft soil for embankment support, an important stress concentration develops in the stone. Stress concentration in the stone column results in an increase in resisting shear force that must be taken advantage of for an economical design. In performing hand stability computations, vertical stress concentration can be easily handled using equations (8a) and (8b) without special modifications.

In computer analyses, the effects of stress concentration can be

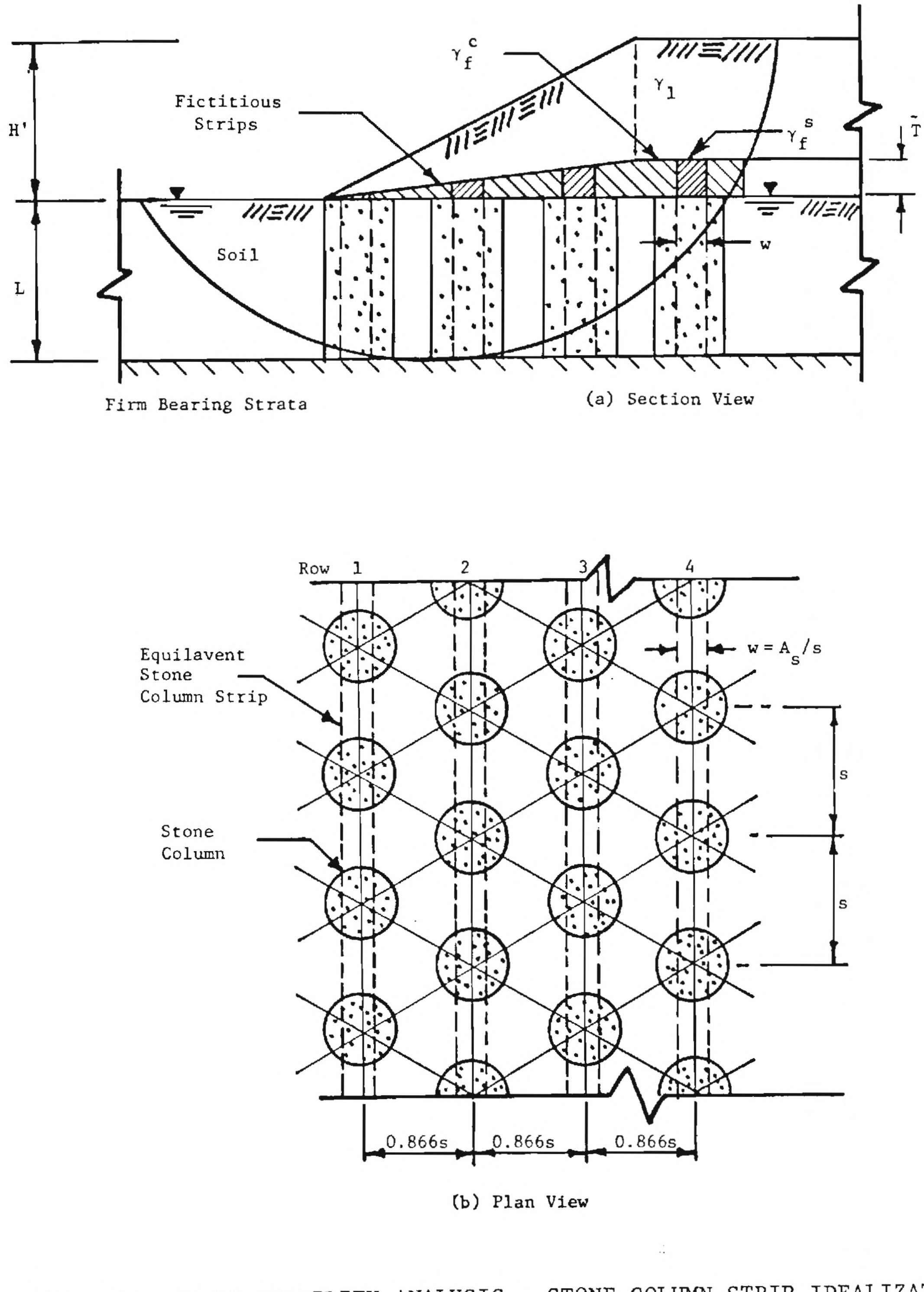


FIGURE 46. SLOPE STABILITY ANALYSIS - STONE COLUMN STRIP IDEALIZATION AND FICTITIOUS SOIL LAYER (FRICTIONLESS SOIL).

handled by placing thin, fictitious strips of soil above the foundation soil and stone columns at the embankment interface (Fig. 46). The weight of the fictitious strips of soil placed above the stone is relatively large to cause the desired stress concentration when added to the stress caused by the embankment. The weight of fictitious soil placed above the in-situ soil must be negative to give the proper reduction in stress when added to that caused by the embankment. The fictitious soil placed above both the stone columns and in-situ soil would have no shear strength.

The average vertical stress σ acting at the interface between the embankment and the stone column reinforced ground is usually assumed to be equal to the height of the embankment H' at that location times its unit weight γ_1. Let the stress concentration in the stone column be composed of the following two parts:

$$\sigma_s = \sigma + \Delta\sigma_s \tag{31}$$

where $\sigma = \gamma_1 H'$ is the usual stress due to the embankment fill and $\Delta\sigma_s$ is the stress that must be added to σ to give the correct stress concentration in the stone column. Rearranging equation (31) gives:

$$\Delta\sigma_s = \sigma_s - \sigma = \mu_s\sigma - \sigma = (\mu_s - 1)\sigma \tag{32a}$$

which simplifies to

$$\Delta\sigma_s = (\mu_s - 1)\gamma_1 H' \tag{32b}$$

Now let the thickness of the fictitious layer be $\tilde{T}$ and the unit weight above the stone be γ_f^s and above the soil γ_f^c. From equation (32b) the fictitious weight of the soil above the stone must be:

$$\gamma_f^s = (\mu_s - 1)\gamma_1 H'/\tilde{T} \tag{33}$$

Similarly the fictitious layer above the soil must have a unit weight of

$$\gamma_f^c = \frac{(\mu_c - 1)\gamma_1 H'}{\tilde{T}} \tag{34}$$

where μ_s and μ_c are given by equations (8a) and (8b).

When stress concentration is considered to be present (i.e., $n \neq 1$), equations (33) and (34) are employed to calculate the equivalent weight of the fictitious strips. The equivalent thickness $\tilde{T}_i$ of each strip should be made small to avoid changing the geometry of the problem. Use of a constant thickness of 0.25 to 0.5 ft. (75-100 mm) is suggested above both the stone and soil beneath the full height of embankment, with the thickness tapering to zero at the toe as illustrated in Fig. 46. The fictitious strips must also have no shear strength (or else a very small value). The driving moments caused by the fictitious strips are cancelled out since the stress concentration does not change the total force exerted by the fill. *Finally,* *either individual circles should be checked, or else limits placed on the radius and/or the grid size of circle centers to be checked so that the critical circle is not controlled by the weak, fictitious interface layer.*

Average Shear Strength Method

The average shear strength method is widely used in Japan to analyze the stability of sand compaction piles [24,66] and has been used more recently in the U.S. [9]. In this method the weighted average material properties are calculated for the material within the unit cell. The soil having the fictitious weighted material properties is then used in a stability analysis. It is important to remember that stone columns must actually be located over the entire zone of material having weighted shear properties through which the circular arc passes. Since average properties can be readily calculated, this approach is appealing for both hand and computer usage. However, as discussed subsequently, average properties cannot in general be used in standard computer programs when stress concentration in the stone column is considered in the analysis.

Hand Calculation. Stress concentration can be readily included in hand stability calculations using the weighted shear strength method. Consider the general problem of stone column reinforced ground where the stone column has only internal friction ϕ_s, and the surrounding soil is undrained but has both cohesion c and internal friction ϕ_c. The stress state within a selected stone column unit cell is shown in Fig. 47 at a depth where the circular arc intersects the centerline of the stone column. The effective stress in the stone column due to the weight of the stone and applied stress σ can be expressed as:

$$\bar{\sigma}_z^s = \bar{\gamma}_s z + \sigma\mu_s \tag{35}$$

where: $\bar{\sigma}_z^s$ = vertical effective stress acting on the sliding surface of a stone column
$\bar{\gamma}_s$ = unit weight of stone (use bouyant weight below the ground water table)
z = depth below the ground surface
σ = stress due to the embankment loading (usually taken as the stress at the embankment-ground interface)
μ_s = stress concentration factor for the stone column, equation (8b)

The shear strength of the stone column neglecting cohesion is then expressed as

$$\tau_s = (\bar{\sigma}_z^s \cos^2\beta)\tan\phi_s \tag{36}$$

where: τ_s = shear strength in the stone column
β = inclination of the shear surface with respect to the horizontal
ϕ_s = angle of internal friction of the stone column

The total stress in the cohesive soil considering stress concentration

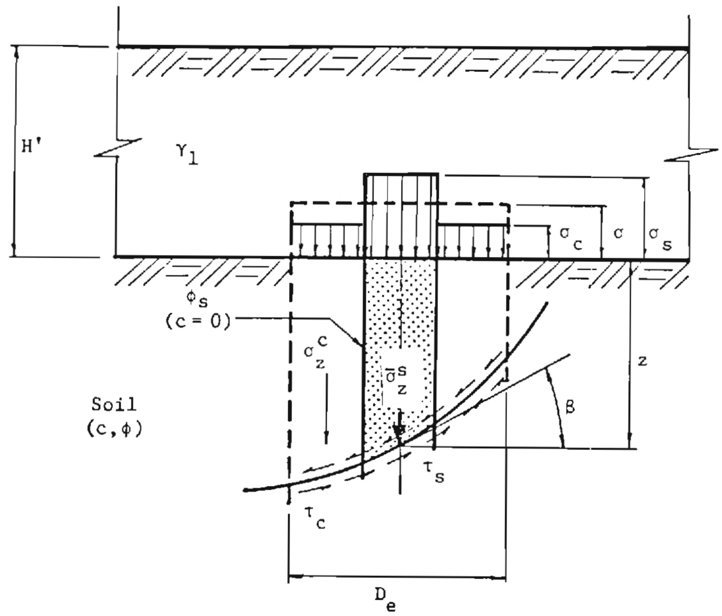


FIGURE 47. NOTATION USED IN AVERAGE STRESS METHOD STABILITY ANALYSIS.

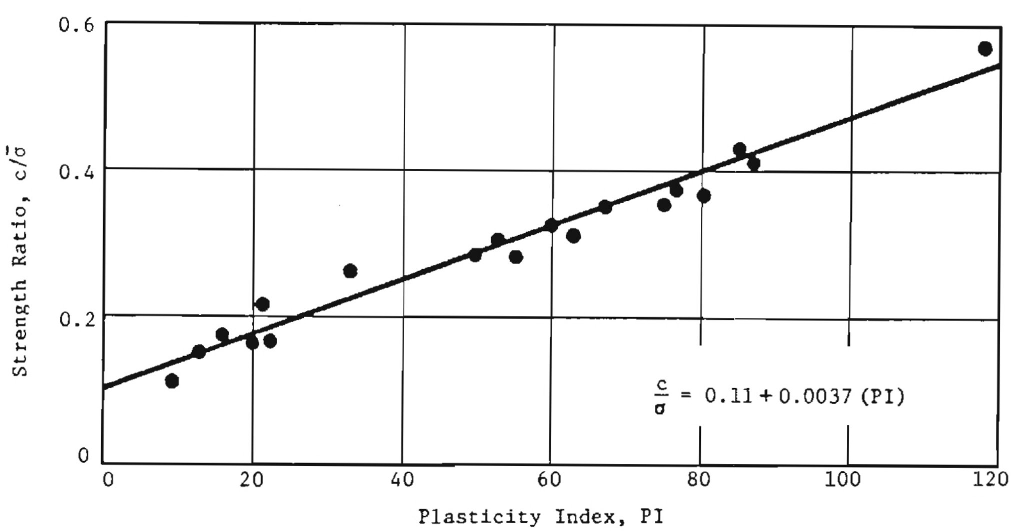


FIGURE 48. VARIATION OF SHEAR STRENGTH RATIO $c/\bar{\sigma}$ WITH PLASTICITY INDEX - NORMALLY CONSOLIDATED CLAY [133].

becomes

$$\sigma_z^c = \gamma_c z + \sigma \mu_c \tag{37}$$

where: σ_z^c = total vertical stress in the cohesive soil

γ_c = unit weight of cohesive soil

and the other terms were defined above. The shear strength of the cohesive soil is then

$$\tau_c = c + (\sigma_z^c \cos^2\beta)\tan\phi_c \tag{38}$$

where: τ_c = undrained shear strength of the cohesive soil

c = cohesion of cohesive soil (undrained)

ϕ_c = angle of internal friction of cohesive soil (undrained)

The average weighted shear strength τ within the area tributary to the stone column is

$$\tau = (1 - a_s)\ \tau_c + a_s\ \tau_s \tag{39}$$

where all terms have been defined above.

The weighted average unit weight within the reinforced ground is used in calculating the driving moment

$$\gamma_{avg} = \gamma_s \cdot a_s + \gamma_c a_c \tag{40}$$

where γ_c and γ_s are the saturated (or wet) unit weight of the soil and stone, respectively. In this approach the weighted shear strength and unit weight are calculated for each row of stone columns and then used in a conventional hand analysis.

No Stress Concentration. If stress concentration is not present, as is true in some landslide problems, a standard computer analysis can be performed using a conventional program and average shear strengths and unit weights. Neglecting cohesion in the stone and stress concentration, the shear strength parameters for use in the average shear strength method are

$$c_{avg} = c \cdot a_c \tag{41}$$

$$[\tan\phi]_{avg} = \frac{\bar{\gamma}_s a_s \tan\phi_s + \gamma_c a_c \tan\phi_c}{\bar{\gamma}_{avg}} \tag{42}$$

where $\bar{\gamma}_s$ is the bouyant unit weight (if below the groundwater table), and $\bar{\gamma}_{avg}$ is given by equation (40) using the bouyant weight for $\bar{\gamma}_s$ and saturated weight for γ_c (undrained shear).

Use of $[\tan\phi]_{avg}$ based just on the area ratio [9] is not correct as can be demonstrated by considering the case when $\phi_c = 0$. If averages based on area were used then

$$[\tan\phi]_{avg} = a_s \tan\phi_s \tag{43}$$

which would be appropriate to use if $\gamma_{avg} = \bar{\gamma}_s$, but incorrect if the $\bar{\gamma}_{avg}$ used is that required to give proper driving moments, equation (40).

Lumped Moment Method. The lumped moment method can be used to determine the safety factor of selected trial circles by either hand or with the aid of a computer. Following this approach the driving moment M_d and resisting moment M_r are calculated for the condition of no-ground improvement with stone columns. The correct *excess* resisting moment ΔM_r and driving moment ΔM_d due to the stone columns are then added to the previously calculated moments M_r and M_d, respectively. The safety factor of the improved ground is then calculated by

$$SF = (M_r + \Delta M_r)/(M_d + \Delta M_d) \tag{44}$$

In general this approach is most suited for hand calculation. The approach can also be used with computer programs which permit adding in ΔM_r and ΔM_d which could be calculated by hand. This general approach including example problems has been described in detail elsewhere [92].

INCREASE IN SHEAR STRENGTH DUE TO CONSOLIDATION

The shear strength of a soft cohesive soil increases during and following construction of an embankment, tank, or foundation on soft cohesive soils. The additional stress due to construction results in an increase in pore pressure causing consolidation accompanied by an increase in shear strength. The rate of construction of embankments is frequently controlled to allow the shear strength to increase so that the required safety factor with respect to a stability failure is maintained.

The undrained shear strength of a normally consolidated clay has been found to increase linearly with effective overburden pressure [88] as illustrated in Fig. 48. For this type cohesive soil the undrained shear strength can be expressed as

$$c = K_1 \times \bar{\sigma} \tag{45}$$

where: c = undrained shear strength
$\bar{\sigma}$ = effective overburden pressure
K_1 = the constant of proportionality defining the linear increase in shear strength with $\bar{\sigma}$, $K_1 = c/\bar{\sigma}$

For a cohesive soil having a linear increase in shear strength with $\bar{\sigma}$, the increase in undrained shear strength Δc_t with time due to consolidation can be expressed for stone column improved ground as

$$\Delta c_t = K_1 \cdot (\sigma\mu_c) \cdot U \tag{46}$$

where: Δc_t = increase in shear strength at time t of the clay due to consolidation

σ = average increase in vertical stress in the unit cell on the shear surface due to the applied loading

μ_c = stress concentration factor in the clay, equation (8a)

U = degree of consolidation of the clay at time t

Equation (46) gives a convenient method for estimating the increase in shear strength in the cohesive layer at any time provided K_1 has been evaluated from field testing. The applied stress σ considers the embankment loading $\gamma_1 H'$ and is reduced, if required, to consider the spreading of stress in the stone column improved ground as discussed in a previous section.

DISCUSSION

Bearing Capacity

Several theories were presented for predicting the ultimate bearing capacity of an isolated, single stone column. The Vesic cavity expansion theory, equation (14), is the most widely used. Frequently in practice, interaction between stone columns is neglected, and the calculated capacity of an isolated single column is assumed equal to the capacity of each column within a group. A slightly better estimate of ultimate capacity would be obtained by increasing the capacity of an isolated stone column using the shape factors shown in Fig. 8 as a guide. The group bearing capacity theory presented in this chapter offers an alternate approach for predicting the ultimate capacity of groups although further experience is needed using this approach. Finally, a circular arc stability analysis is commonly used in practice to estimate the stability under the edge of a wide group of stone columns such as occurs under an embankment or tank type loading. Stability analyses are also used to evaluate the beneficial effects of stabilizing landslides using stone columns.

Full-Scale Load Test Results. Bearing capacity factors backfigured from the results of full-scale, field load tests performed on both single, isolated stone columns and groups of stone columns are compared, in Fig. 49, to theoretical values for isolated columns obtained from cavity expansion theory. The backfigured bearing capacity factors are arbitrarily shown on this figure on the vertical line $\phi_s = 42°$ to be able to compare theory and observed values; no assumption was made concerning ϕ_s in backfiguring the value of $\tilde{N}_c$. The bearing capacity of the soil is considered in figuring $\tilde{N}_c$ (Fig. 49).

The field test results indicate a single stone column has a bearing capacity factor $\tilde{N}_c$ between about 20 and 27. Measured bearing capacity factors for stone columns within large groups vary from about 15 to 28. In

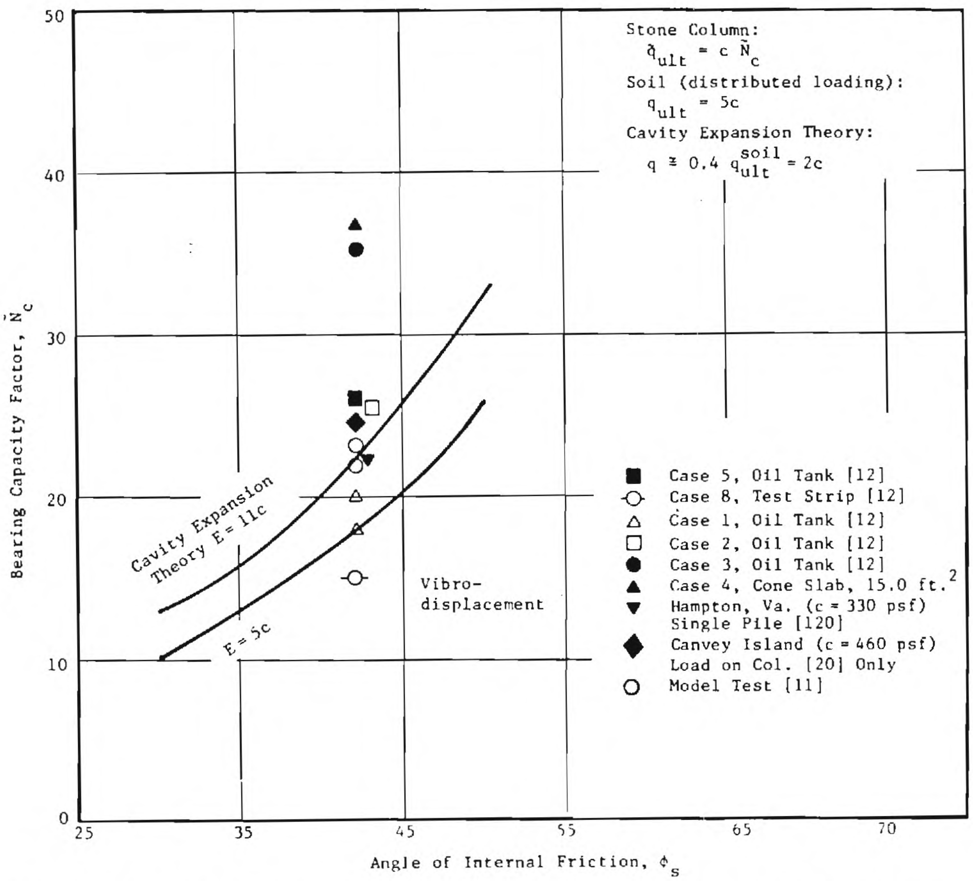


FIGURE 49. COMPARISON OF CALCULATED AND MEASURED BEARING CAPACITY FACTORS FOR STONE COLUMN IMPROVED GROUND.

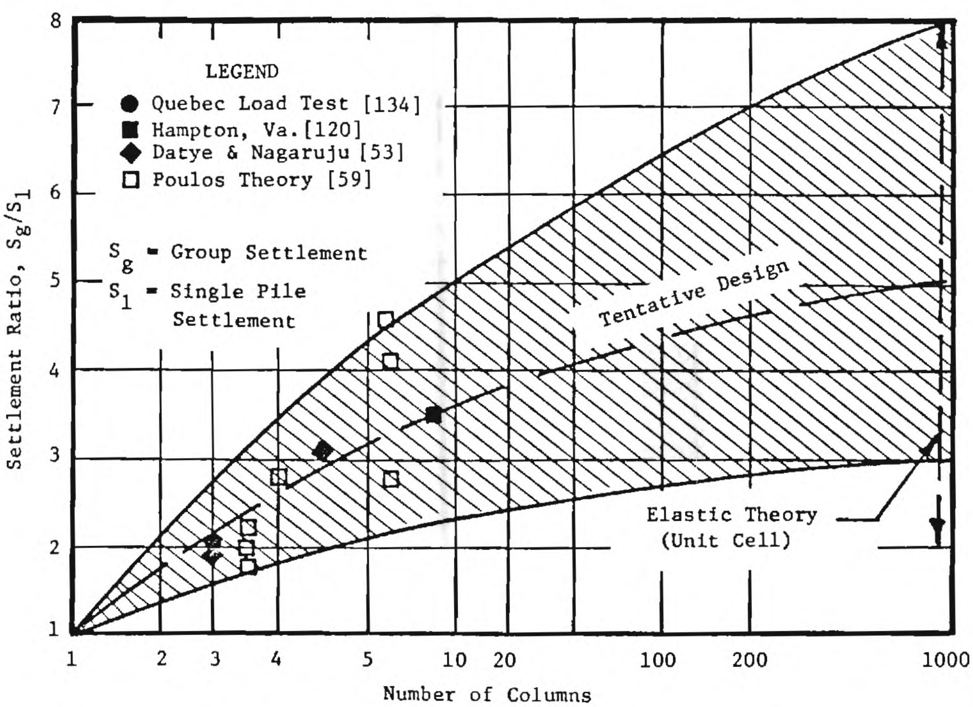


FIGURE 50. GROUP SETTLEMENT AS A FUNCTION OF NUMBER OF STONE COLUMNS: $s \cong 2D$.

this analysis the stress carried by the soil was taken to be 5c which was always equal to or less than σ_c, equation (8a).
Undoubtedly the backfigured bearing capacity factors for the tanks reflect some increase in strength due to consolidation as the load is applied; construction rates are not known.

In Fig. 49 the upper limit for the cavity expansion theory is defined by E = 11c and the lower limit by E = 5c. This range of moduli approximately bounds the observed bearing capacity factors for stone columns in groups.

Settlement Predictions

Large Groups. One approach for predicting primary consolidation settlement of a wide group of stone columns resting on a firm stratum is to use elastic finite element theory for low compressibility soils (Figs. 24 to 26) or nonlinear finite element theory for high compressibility soils (Figs. 29 to 37). To predict long-term primary consolidation settlements the drained modulus of elasticity of the cohesive soil must be used. If drained triaxial tests have not been performed, the drained modulus of elasticity of the cohesive soil can be calculated from the results of one-dimensional consolidation tests using [62]:

$$E = \frac{(1+\nu)(1-2\nu)(1+e_o)\sigma_{va}}{0.435\ (1-\nu)\ C_c} \tag{47}$$

where: E = drained modulus of elasticity (for a stress path along the K_o line)
e_o = initial void ratio
C_c = compression index
ν = Poisson's ratio (drained)
σ_{va} = average of initial and final stress state applied in the field (vertical stress)

The modulus of elasticity E given by equation (47) is a general material parameter and can be used for three-dimensional settlement problems if properly selected. The primary limitation in estimating E from equation (47) is the ability to choose the correct value of Poisson's ratio, since E is very sensitive to the value of the ν used. Recommended ranges of Poisson's ratio are given in Chapter VII.

A sample of material in a one-dimensional consolidation test cannot deform in the lateral direction. For a condition of no lateral movement, the constrained elastic modulus $\tilde{D}$ is equal to

$$\tilde{D} = E(1-\nu)/[(1+\nu)(1-2\nu)] \tag{48}$$

where E is the modulus of elasticity and ν is Poisson's ratio. The constrained elastic modulus is defined as the vertical stress divided by the vertical strain for the condition of one-dimensional settlement (i.e., no lateral movement). Since the unit cell idealization is somewhat similar to the one-dimensional consolidation test, the moduli E and $\tilde{D}$ tend to give a simple bound on settlements when used in equations such as (23).

Settlement of Limited Groups. For stone column groups less than about 20 to 40 columns the methods for estimating settlement using the unit cell idealization are overly conservative. As previously discussed, in groups of limited size, the vertical stress spreads outward from the stone column and decreases with depth. This reduction in stress can be readily considered in the equilibrium, incremental and finite element methods. In these methods the average stress σ within the compressible zone can be estimated using an appropriate stress distribution theory as previously discussed. The vertical settlement can then be calculated as an approximation by applying this average stress to the top of the unit cell idealization of the compressible zone.

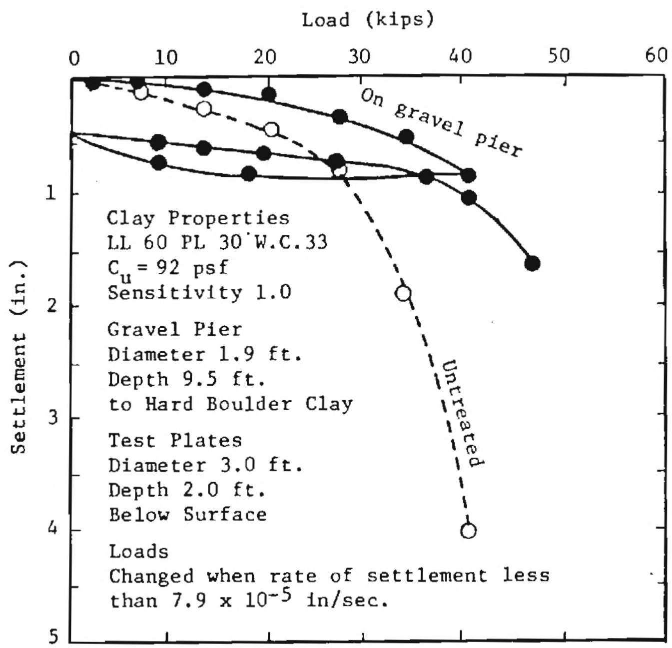
The approximate elastic solution for pile groups given by Poulos [59] has also been used for predicting settlements of small groups [73]. Balaam [57] has extended Poulos' earlier work and developed a series of interaction curves for calculating group settlements of stone columns.

Fig. 50 shows a comparison between observed group settlements and the bounds for typical geometries and material properties used in the Poulos [59] theory. The linear elastic theory developed in this study is also shown on the figure. The linear elastic theory uses the unit cell idealization to model an infinite group of stone columns; a low compressibility soil was assumed having a modular ratio of 10. Both measured and theoretical settlements are expressed as dimensionless settlement ratios of the group settlement to the settlement of a single stone column.

The theories reasonably bound the limited number of measured group settlements. Of practical importance is the finding that a three column group settles about twice as much as a single pile and a seven column group three times as much. Using his interaction curves, Balaam [57] predicted a settlement ratio of about 1.8 compared to the measured value of 3.0 for the 7 column group described by Datye and Nagaruju [53]; the stone columns were constructed by the ramming technique. Group settlements were also underpredicted by Balaam [57] by about 25 percent for a 3 column model group using the interaction factors.

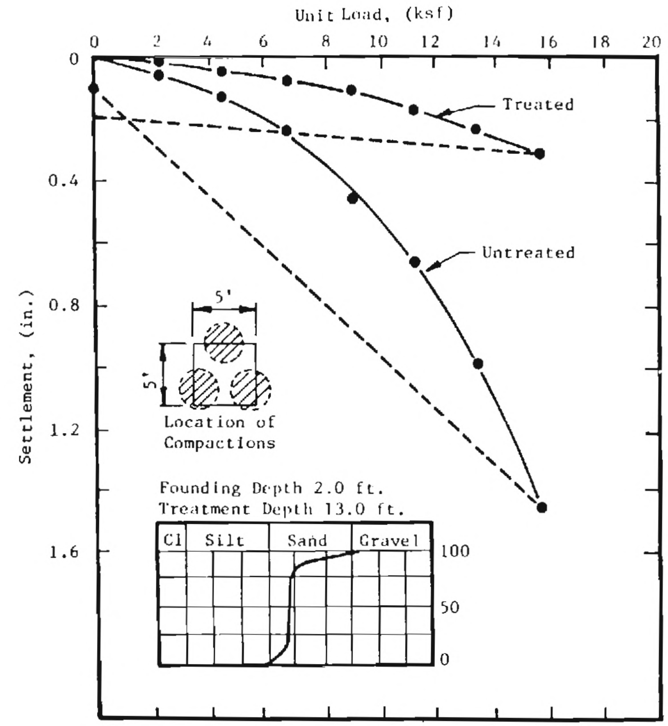
The settlements of a ten column group may be as much as 3 to 4 times or more than of a single pile. Therefore, similarly to a load test performed on a conventional pile, group settlements are appreciably greater than indicated from the results of a single stone column load test. If load tests are performed on single columns or small groups, the results should be extrapolated to consider settlement of the group using Fig. 50 for a preliminary estimate.

The effectiveness of reinforcing soft soils with stone columns to reduce settlement becomes greater with increasing settlement, as the full bulge and resulting passive soil resistance is mobilized. The significant increase in resistance to deformation with increase in load of reinforced ground compared with unreinforced ground is shown in Figs. 51 and 52 for a load test on a single column and a 3 column group, respectively. Figs. 51



(Note: 1 in. = 25.4 mm; 1 kip = 4.45 kN; 1 psf = 0.0479 kN/m^2; 1 ft. = 0.305 m)

FIGURE 51. COMPARISON OF SINGLE STONE COLUMN LOADING TEST RESULTS ON TREATED AND UNTREATED CLAY - EAST GLASCOW [15].



(Note: 1 ksf = 47.9 kN/m^2; 1 in. = 25.4 mm; 1 ft. = 0.305 m)

FIGURE 52. COMPARISON OF GROUP LOADING TEST RESULTS ON TREATED AND UNTREATED SAND - PORT TALBOT [19].

and 52 also show that at working load the undrained response of the columns is reasonably linear for clay and sand. Fig. 53 shows a comparison of settlement of stone column treated ground with untreated ground for large scale load tests.

SUMMARY

Cavity expansion theory can be used with reasonable accuracy to calculate the ultimate capacity of a single, isolated stone column. A general theory to predict group bearing capacity of square and infinitely long rigid foundations was also developed, but needs further verification. In soft soils consideration should be given to the reduction in the combined strength of the soil and stone column for both group bearing capacity and circular arc type failures. Relatively little is known at the present time about the composite behavior of a stiff stone column acting together with a weak soil (refer to Chapter VII).

The equilibrium, finite element and incremental methods can be used to estimate primary consolidation settlement. The equilibrium method is quite simple and has been used in Japan for many years to estimate the settlement of sand compaction piles. Only an appropriate value of stress concentration has to be assumed in this method. Design curves are presented based on the finite element method for estimating settlement of (1) low compressibility soils and (2) compressible soils using linear and nonlinear theory, respectively. Finally, the incremental method uses approximate elastic-plastic theory requiring, for practical purposes, a computer solution. All of these approaches require knowing the compressibility characteristics of the soil. For a cohesive soil the one-dimensional consolidation test can be used to evaluate the compressibility; in a sand the Dutch cone or the standard penetration test can be used.

Consolidation theory was presented for estimating the time rate of primary consolidation settlement considering both radial and vertical drainage and also the effects of smear. Because stone columns act as drains, primary settlement in most cases will occur rapidly when stone columns are used; this, in many applications is an important advantage of stone columns. In some instances, however, primary consolidation may occur almost as quickly without stone columns due to the presence of natural permeable seams and high natural horizontal permeability. A discussion of the evaluation of the drainage characteristics of the layer is given in Chapter IV. Finally, the strength gain due to consolidation can and should be considered in bearing capacity and stability analysis.

Secondary settlements can be quite important in organic soils and some soft clays. A method of calculating secondary settlement was presented based on secondary settlement increasing linearly with the logarithm of time. Because of the rapid occurrence of primary consolidation when stone columns are used, secondary settlement is of greater importance than for conventional construction.

Stone columns can be used to increase the stability of both existing

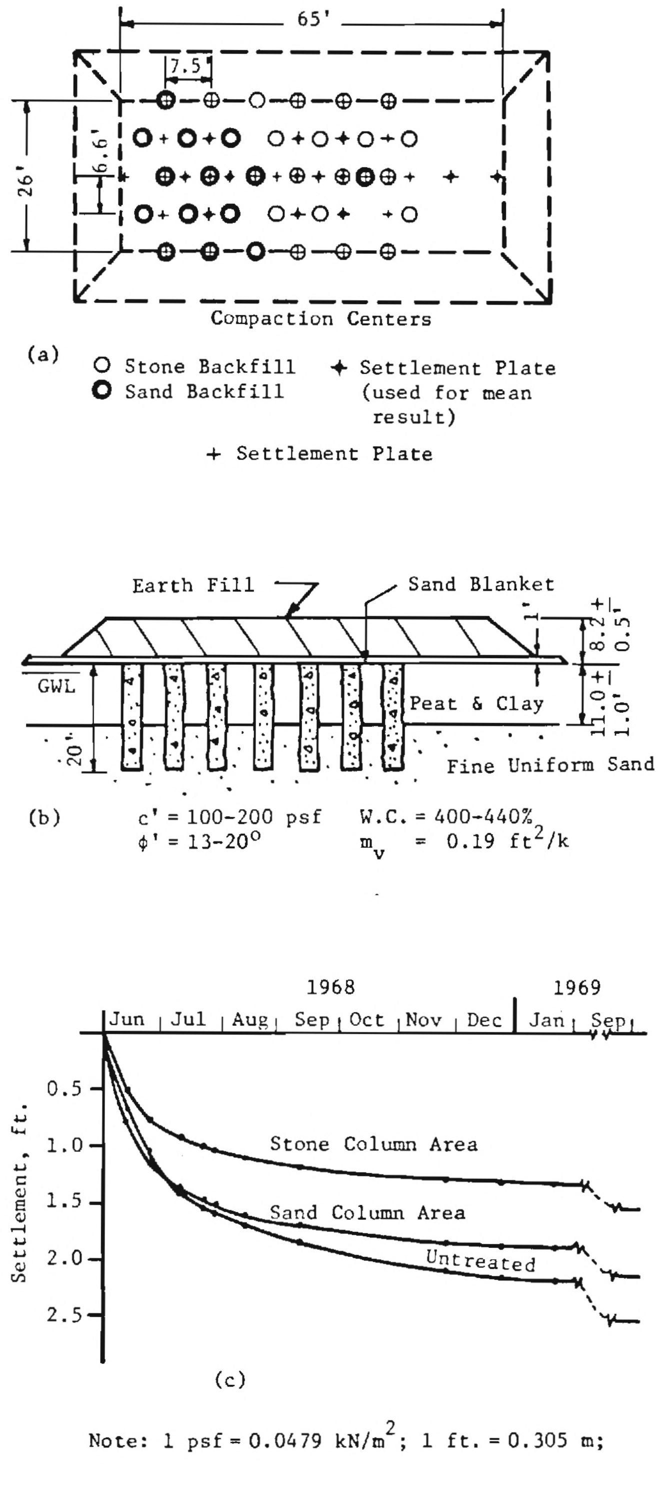


FIGURE 53. COMPARISON OF LARGE SCALE LOADING TEST RESULTS ON TREATED AND UNTREATED CLAY - BREMERHAVEN [19].

slopes and embankments constructed over soft ground. Stability analyses including stress concentration can be performed using conventional computer programs having the capability of LEASE or STABL. The average shear strength or the lumped moment method are both suitable for hand computation. For an economical design, stability analyses must be performed considering, where appropriate, stress concentration in the stone column.

Design aspects and specific recommendations are considered in Chapter VII for bearing capacity, settlement and stability of stone columns. Finally, design examples are presented in the appendices.

CHAPTER IV

SUBSURFACE INVESTIGATION, TESTING AND FIELD INVESTIGATION

INTRODUCTION

Stone columns are a design alternative to conventional foundation considerations at marginal sites requiring ground improvement and for the stabilization of existing landslides. The existence of poor site conditions will often be known or suspected before beginning the subsurface investigation either from local experience or consideration of the type landform. To adequately evaluate design alternatives including stone columns for such marginal sites, a more thorough subsurface investigation and laboratory and field testing program are required than for better sites. Marginal sites for highway projects where stone columns might be considered as a design alternative include:

1. Sites for moderate to high embankments or bridge approach fills underlain by cohesive soils having shear strengths less than 600 to 800 psf (30-40 kN/m^2).

2. Marginal sites for foundations such as bridge abutments and bridge piers underlain by cohesive soils with shear strengths greater than about 600 to 800 psf (30-40 kN/m^2), or very loose to loose silty sands having silt contents greater than about 15 percent.

3. Landslide areas.

A more detailed discussion of stone column applications is given in Chapter I and selected case histories in Chapter VI.

All too often inadequate (inexpensive) subsurface investigations have led to serious problems during and after construction, sometimes accompanied by spectacular failures. As an example, an 18-story building was supported on spread footings overlying ground improved by stone columns. After construction, one corner of the building settled about 12 in. (305 mm) while the rest of the building underwent little movement. As a result of this large differential settlement, the building was structurally damaged and abandoned one year after construction. The foundation design for the building was based on an inadequate subsurface investigation; the owner received what he paid for. To complicate matters the original investigation was carried out for a 4-story structure which was later changed to 18 stories. The original subsurface investigation indicated the presence of about 18 ft. (5.5 m) of sand over rock. After excessive settlements developed a subsequent investigation disclosed the corner of the building showing distress to be underlain by 7 to 8 ft. (2.1-2.4 m) of peat. Had the presence of the peat layer been known, the design could have been modified and distress to the building avoided. Also, field inspection would have detected the presence of the peat during construction and required modifications to the design could have been made.

Properly planning and executing the subsurface investigation and the soil testing program is extremely important to the overall success of selecting an engineering sound, cost-effective design for embankments, abutments, bridge foundations located on marginal soils and landslide stabilization work. An adequate number of test borings must be provided to reliably depict the site conditions including the shear strength and settlement characteristics, the extent of peat and organic deposits, and the occurrence of thin, permeable seams and zones.

The geotechnical properties of soils vary significantly with depth and usually to a lesser extent laterally across a site. This is certainly to be expected since the formation of soil deposits (particularly those in water) are "random in space and time [93]." As a consequence some degree of risk is always involved in geotechnical projects. The risk, however, can be minimized by conducting a thorough subsurface investigation including careful planning, precise execution and good feedback among the engineers, geologists and technicians involved.

SPECIAL CONSIDERATIONS FOR STONE COLUMNS

When stone columns are a possible design alternative the following additional considerations must be integrated into the planning and execution of the subsurface investigation:

1. Peat. As illustrated by the example, the subsurface investigation must locate and fully evaluate either extensive or localized deposits of peat, muck or other organic soils. The presence of such materials can dictate the construction method used to form the stone columns, or even show the site to be unsuitable for this method of ground improvement. The structure of the peat is also an important consideration.

2. Permeable Strata. Stone columns act as vertical drains. For sites having relatively low natural horizontal permeabilities, the use of stone columns can greatly accelerate primary consolidation. Therefore to fully access the potential advantage of stone columns with respect to time-rate of primary consolidation settlement, the vertical and horizontal consolidation characteristics of the soils must be evaluated. The subsurface investigation must determine the presence and extent of thin seams, layers or lenses of permeable soils such as sand, gravel, or shells. Even relatively homogeneous appearing clays may be stratified, having relatively permeable sand and silt layers. If the natural horizontal permeability is sufficiently great, use of stone columns may not accelerate primary consolidation, and one important advantage of stone columns may not exist.

Permeability is important since field tests give permeability rather than the coefficient of consolidation. Also, a reliable value of the horizontal coefficient of consolidation cannot be determined using a consolidation test if the soil has permeable seams. Rather, horizontal permeability must be determined in either the field or laboratory, and the coefficient of consolidation calculated.

3. Stability. A primary use of stone columns beneath embankments is to provide an adequate margin of safety with respect to overall embankment stability. Therefore, the evaluation of a representative shear strength of both the foundation soil and the stone column are important. Soft zones or thin soft layers of cohesive materials or thin sand seams in which pore pressures may build up can have a dominant effect on the overall stability of an embankment. For embankments, the short-term (undrained) shear strength will generally control the design.

   For landslide problems, the long-term (drained) shear strength is usually critical. In this type problem the occurrence and movement of water is an important concern.

4. Settlement. Since immediate settlement is complete by the end of construction, it is generally of little practical significance for embankment and approach fill design. Both primary consolidation and secondary compression type settlements are often of significance in the soils in which stone columns may be used for embankment, approach fill, and abutment support. Frequently sites requiring ground improvement involve organic soils or soft clays. For these type soils secondary compression settlement can be as important as primary consolidation. Hence, secondary compression requires special consideration in evaluating the geotechnical properties and in design. Of course, differential settlements between approach fills and bridges is always an important concern.

SUBSURFACE INVESTIGATION

The results of a reconnaissance survey should be used to develop a preliminary indication of the subsurface conditions at the site. At this time, the engineer(s) responsible for the subsurface investigation should, on a preliminary basis, consider potentially feasible design alternatives, such as stone columns for the embankment, approach fill, and bridge support.

For marginal sites where stone columns are considered, the design engineer should be briefed on a regular basis concerning the findings of the subsurface investigation. Further, on a large project samples should be tested as soon as possible so that the results can be given to the geotechnical design engineer [74]. Using the field and laboratory findings, the designer should, on a preliminary basis, tentatively evaluate design alternatives. Frequently the selection of potential design alternatives, as dictated by the site conditions as they are understood at the time, will indicate that important modifications and/or changes are necessary to the subsurface exploration program. All too frequently, the subsurface investigation is complete before design alternatives are even considered. Then it is too late to tailor the investigation to the special improvement techniques that may be necessary such as stone columns.

Soil Profile

An erratic soil profile is frequently present at sites where stone columns are a potential design alternative. Also, the occurrence of peat layers or pockets and thin layers or lenses of very soft clays are of great importance in evaluating the use of stone columns for either embankment or bridge support. The emphasis in the subsurface investigation should therefore be placed on establishing the full variation in the soil properties rather than running a large number of laboratory tests on samples selected at random [93]. In other words, put down a sufficient number of test borings and conduct sufficient vane shear tests to establish the likely extreme variation in site conditions.

In very soft to firm cohesive soils, the vane shear test should be used to establish the variability of the soil profile. The vane shear test is easy to perform and gives a reliable definition of the shear strength profile. In soft and very soft cohesive soils, the standard penetration test is not sensitive enough to be of practical use. In marginal very loose to firm silty sands, the standard penetration test or preferably the Dutch cone can be used to define the soil profile at bridge sites. If the Dutch cone is used standard penetration testing should also be performed to obtain split spoon samples of the material.

Frequently relatively thin, soft cohesive deposits are found overlying sands which was the case at the Hampton, Virginia stone column site [27]. For such conditions, once the extreme variability of the cohesive soils has been established, soil test borings should be performed beside selected vane test locations where the extreme and average conditions are encountered. Standard penetration testing should be performed at least in the cohesionless soils and jar samples saved for both the cohesive and cohesionless strata. Undisturbed samples should also be taken at these locations to determine the consolidation, shear strength and permeability characteristics of the soft soils as subsequently discussed. A stationary piston sampler should be used having a 3 in. (76 mm) minimum diameter and a thin wall. Continuous tube sampling should be performed at selected locations within the soft, cohesive strata to aid in determining if thin sand layers, seams, or partings are present.

The vane shear tests and the standard penetration tests in the test borings should be performed at a 5 ft. (1.5 m) interval. For depths greater than 30 to 40 ft. (9-12 m) consideration could be given to increasing the interval to 10 ft. (3 m). If a thin stiff upper crust is present due to dessication, the testing interval should be reduced to 2.5 ft. (0.8 m) to define this strata. The ground water level should be determined in the test borings at the time of the boring and also 24 hours later. For some special applications of stone columns such as slope stabilization, the ground water level and its variation with time is likely of critical important. In this case piezometers should be set in selected holes for long-term observations.

Test Pits - Peat and Desiccated Layer

If peat is encountered the type structure and fabric of the peat should be determined where stone columns are a design alternative. Open test pits should be used, where feasible, for peat deposits located at or near the surface. If test pits are not feasible, undisturbed samples should be obtained and inspected. A shallow test pit may also be desirable to investigate the structure of the dessicated crust, if present. Flaate and Preber [96] have pointed out that when embankments fail sometime after construction, failure is usually at least partially related to a gradual weakening of the weathered crust. Due to the embankment weight, the dessication cracks open resulting in softening of the soil with time in the vicinity of the crack and a reduction in shear strength. Therefore, a conservative estimate of the shear strength of the crust should be used in a stability analysis.

Test Borings

A detailed consideration of subsurface exploration including sampling and the depth and spacing of test borings has been given by Hvorslev [94]. Refer to this important reference for this aspect of the investigation.

For embankments to be placed on marginal sites, the very soft cohesive layers often found near the surface are likely to control the performance with respect to both settlement and stability. Therefore, the majority of test borings need only extend 15 ft. (3 m) into a competent strata, *provided sufficient deeper test borings are performed to verify weaker strata are not present below this level.*

For performing a subsurface investigation, the vertical stress in a large group of stone columns should be considered concentrated within the stone column group down to the tips of the columns. Therefore, depending upon the length of the stone columns the test borings must be carried down a sufficient depth below the tip to avoid either stability or settlement problems. Similar concepts for determining the required depth of borings apply to stone columns as for shallow or deep foundations [97]. Settlements can be caused below the tips of stone columns supporting bridge piers and abutments to a depth below the stone columns where the change in stress due to the construction is equal to 10 percent of the initial effective stress. The same general concept applies for embankments supported on stone columns. For embankments, however, the 10 percent criteria is probably somewhat severe considering that larger settlements are usually tolerable for embankments than for bridge foundations. For embankments supported on stone columns the required depths below the tip of the stone column even relaxing the requirements to 15 to 20 percent will generally be great. Therefore realistic boring depths should be selected considering both stress changes and the geology of the site, together with sound engineering judgement.

MATERIAL PROPERTIES

Selection of the best design alternative is dependent upon accurately establishing the subsurface conditions and determining material properties representative of the in-situ soil. In determining reliable material properties, the obtaining of representative samples and the evaluation of sample disturbance are important, but often overlooked, factors that cannot be emphasized too much. Every step in the sampling, extrusion, and trimming processes causes varying degrees of sample disturbance. Several approximate techniques, however, are available that can be used to at least approximately account for disturbance.

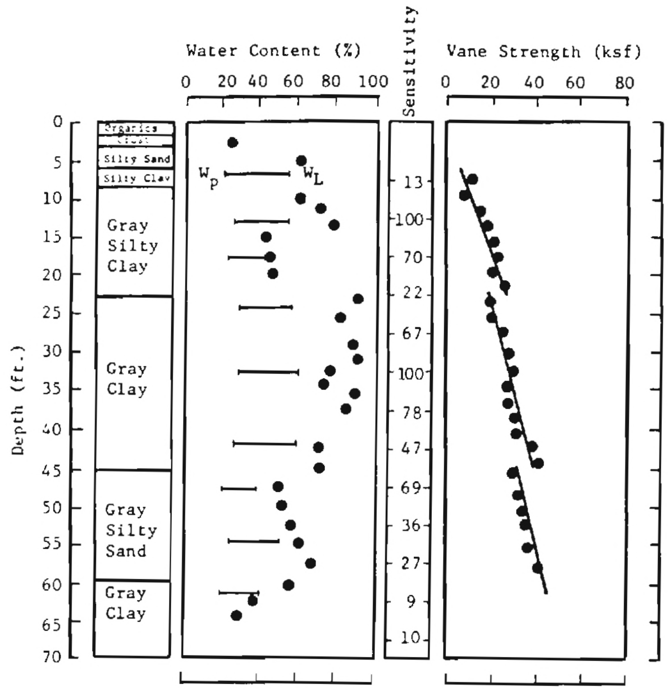
Index Properties

The simple index tests, when properly interpreted, can give important information concerning the variability and past history of the deposit. Therefore, the liquid limit, plastic limit, and in-situ water content of the cohesive soils should be determined. A natural water content near the liquid limit indicates a normally consolidated soil; a water content near the plastic limit indicates preconsolidation. In general the soil should not be allowed to dry out before evaluating the water content and Atterberg limits. This index data, together with the shear strength characteristics (including sensitivity) of the cohesive layer(s) and standard penetration resistance or cone resistance of cohesionless layers should be summarized in the form of a boring log and index property profile as illustrated in Fig. 54.

In slightly marginal very loose to firm coehsionless soils, vibrofloation, stone columns and dynamic consolidation may be design alternatives. If the silt content is greater than 15 percent, densification by vibrofloation should not be considered. For such silty soils stone columns and dynamic consolidation would be design alternatives for slightly marginal sites. Therefore, for such sites where preliminary results indicate the silt content to be between about 10 and 25 percent, a relatively large number of grain size tests (washed through the No. 200 sieve) should be performed to aid in determining which improvement methods are feasible. Where cohesive soils are predominant, grain size tests should also be performed on the cohesionless seams and layers to aid in estimating the permeability of the strata. Horizontal permeability is important in estimating the time rate of consolidation of cohesive soils as discussed previously.

Shear Strength of Cohesive Soil

The undrained shear strength of the cohesive soil should be used for performing stability analyses during and at the end of construction of stone column supported structures. The field vane test is the best method for evaluating the undrained shear strength of very soft to firm cohesive soils. The shear strength obtained from field vane tests, however, should be corrected to reflect back calculated shear strengths as discussed subsequently in Chapter VII. The unconsolidated-undrained triaxial test, unconfined



(Note: 1 ksf = 47.9 kN/m^2; 1 ft. = 0.305 m)

FIGURE 54. TYPICAL PRESENTATION OF FIELD RESULTS FOR SENSITIVE CANADIAN CLAYS [110].

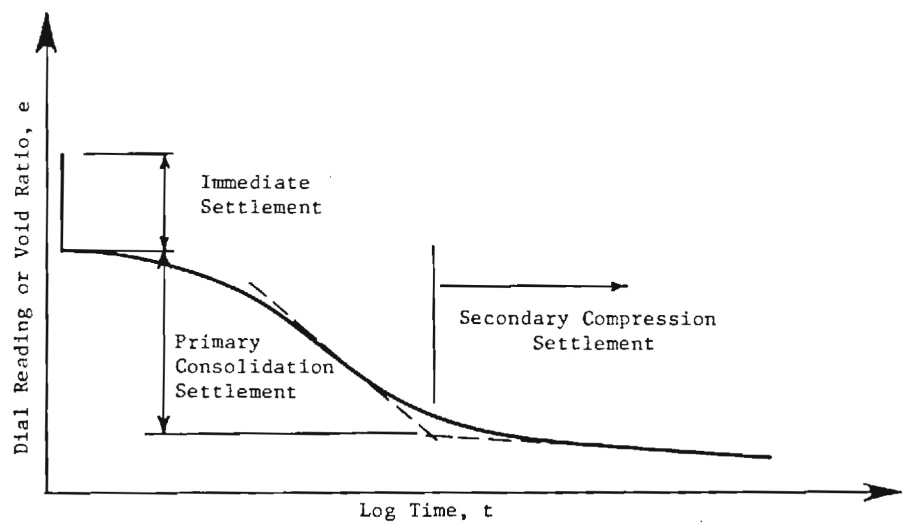


FIGURE 55. COMPONENTS OF SETTLEMENT OBTAINED FROM ONE-DIMENSIONAL CONSOLIDATION TEST.

compression test, and laboratory vane test can also be used to evaluate the strength of cohesive soils. The effects of sample disturbance on tube samples can in many instances be reduced, at least when compared to the disturbance affects on unconfined compression test results, by running an unconsolidated-undrained test in a triaxial cell using a representative confining pressure. The unconsolidated-undrained triaxial test is therefore generally recommended over the unconfined compression test. A consolidated-undrained shear test can be used to evaluate the undrained strength in soils which normalize using the special technique described by Ladd and Foott [102].

For landslide problems, the long-term stability is usually most critical. For this condition consolidated-undrained shear tests should be performed with pore pressures being measured during the test.

Consolidation Test. The settlement characteristics of the cohesive soils should for convenience be performed in a one-dimensional consolidometer. Recommended details for performing the test so as to minimize sample disturbance have been given elsewhere [98]. In very soft to soft cohesive soils and peats, use of the one-dimensional consolidometer test may significantly underpredict the amount of vertical settlement due to lateral consolidation, bulging and spreading [82].

The consolidation test should be performed on a sufficient number of samples to establish a reasonably valid statistical variation of the settlement characteristics of the compressible layers. For many problems where stone columns are an alternative, the weak zone of most significance with respect to settlement will be reasonably well defined. The field vane shear strength may, however, vary from very soft to even firm within the stratum. The number of tests required depends upon a large number of factors related to the specific project. As a very general guide a minimum of about 10 consolidation tests should be performed in this layer. Fewer tests can be performed in less compressible strata, with the number of tests performed being related to the compressibility of the layer.

In soft clays and organic soils secondary compression settlement can be as important as primary consolidation. Therefore the load must be left on the specimen past the primary consolidation phase for at least one log cycle of time to define the secondary compression characteristics. Secondary compression response need not be obtained for every load increment. Care should, however, be exercised to measure the secondary compression characteristics for the load increments near the stress ranges applicable to the particular problem.

It is often common in practice to include all consolidometer settlement in developing e-logσ plots. Such practice automatically lumps immediate and secondary settlement with primary consolidation settlement. In developing e-logσ curves for primary consolidation where time rate of settlement estimates and/or secondary settlement is of importance, the immediate and secondary settlements should not be included when reducing data for e-logσ plots. Only the primary consolidation settlement shown in Fig. 55 should be used in developing the e-logσ relationship. For either embankment construction or bridge foundation support immediate settlement should be considered separately [72]. Inclusion of immediate and/or secondary settlement with primary consolidation settlement in developing e-logσ curves introduces errors in the magnitude of primary settlement and subsequent estimations of time-rates of settlement.

To obtain the correct consolidation curve, each load increment must be left on until 100 percent of primary consolidation is achieved. If secondary compression settlement is of importance the load must be left on for at least one log cycle of time past the end of primary consolidation. The end of primary consolidation should be obtained using the Casagrande log-time method, while the beginning of primary consolidation can probably be best determined using the Taylor square root of time method [88]. These methods are described in standard soil mechanics references [88,98].

Time Rate of Settlement

For embankment construction over marginal sites the selection of the best design alternative is often, to a large extent, dependent upon the ability to reliably predict the time rate of consolidation settlement associated with each alternative. A reliable estimate of the time rate of consolidation depends upon accurately determining the location of the drainage layers and evaluating the in-situ coefficient of vertical and horizontal consolidation, c_v and c_{v_r}. The vertical coefficient of consolidation c_v and vertical permeability k_v are related as follows:

$$c_v = k_v/(\gamma_w m_v) \tag{49a}$$

where γ_w is the unit weight of the pore water, and m_v is the vertical coefficient of compressibility determined from the consolidation test. Likewise, the horizontal coefficient of consolidation c_{v_r} is related to the horizontal permeability k_h by

$$c_{v_r} = (k_h/(m_v\gamma_w) = c_v(k_h/k_v) \tag{49b}$$

where all the terms have been previously defined. Hence, the coefficients of consolidation c_v and c_{v_r} can be easily evaluated using the above two equations if the horizontal and vertical permeability is determined from field or laboratory tests.

In many soils such as soft clays, peats, and organic clays in which stone columns may be constructed, the horizontal permeability is likely to be 3 to 10 or more times the vertical permeability. The greater permeability is caused by natural stratification, laminations and thin partings of permeable soils. Reliably estimating the time rate of primary settlement in such soils is extremely difficult due to our inability to both identify effective drainage layers and evaluate the in-situ permeability of the strata.

Case History - Field Evaluation of Permeability

A feeling for the problem of estimating the in-situ permeability of an anisotropic soil can be obtained by briefly reviewing a case history described by Casagrande and Poulos [99] comparing field and laboratory permeabilities of a varved clay. The site, located on the New Jersey Turnpike, consisted of about 3 to 15 ft. (0.9-4.6 m) of sensitive, decomposed peat overlying 4 to 25 ft. (1.2-7.6 m) of stiff varved clay having pockets of fine sand and organic silt. Below this, a 25 to 70 ft. (7.6-21 m) stratum was encountered of soft to firm, sensitive varved clay extending down to a depth of about 100 ft. (30 m).

Field pumping tests were conducted using two 14 in. (356 mm) diameter wells, one constructed by jetting a pipe down, and the other by driving a closed end pipe. The wells were filled with sand and sealed at the top with a bentonite-sand mixture. An educator pump was located in the bottom of each well to lower the water. Piezometers and well points were used to measure pore pressures at distances of 15, 30 and 100 ft. (5, 9, and 30 m) from the wells. Both falling head and rising head permeability tests were also carried out in the piezometers. Vertical and horizontal permeability was also measured in the laboratory on 2 in. (51 mm) cube specimens trimmed from 3 in. (76 mm) diameter tube samples. The vertical and horizontal permeability were obtained on the same sample by rotating it 90°.

The results of these tests are summarized in Fig. 56. The pumping tests in the jetted holes gave horizontal permeabilities one to two orders of magnitude greater than the field piezometer and well point tests. The piezometer tests gave horizontal permeabilities equal to or greater than the laboratory tests which showed the lowest permeabilities. Due to disturbance and smear effects, the driven well had a permeability 10 times lower than the jetted well. Also, permeabilities measured in jetted piezometers were 5 to 15 times greater than in driven piezometers.

Permeability Evaluation

On many projects, the evaluation of the vertical and radial coefficient of consolidation using laboratory test results will give an adequate indication of the drainage characteristics. On a few projects, however, the time rate of consolidation will be critical with respect to selecting the most economical alternative which will perform satisfactorily. For such projects field permeability tests should be performed at some time after the initial subsurface investigation, when the critical nature of the consolidation characteristics of the strata become apparent. Field permeability tests are relatively expensive being in the general range of $1,000 to $6,000.

Laboratory Tests. Laboratory tests may indicate permeabilities one or two orders of magnitude less than field tests (Fig. 56). Of course, the larger the size specimen the better the results will be. The conventional one-dimensional consolidation test can be used to evaluate the vertical coefficient of consolidation and hence vertical permeability. Rowe [103] has shown, however, that the coefficient of consolidation obtained from conven-

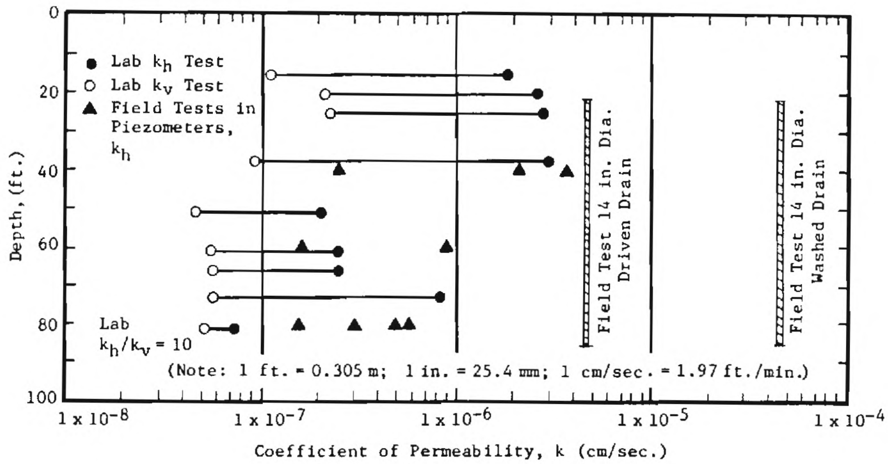


FIGURE 56. COMPARISON OF LABORATORY AND FIELD PUMPING AND PIEZOMETER PERMEABILITY TEST RESULTS IN A VARIED CLAY [99].

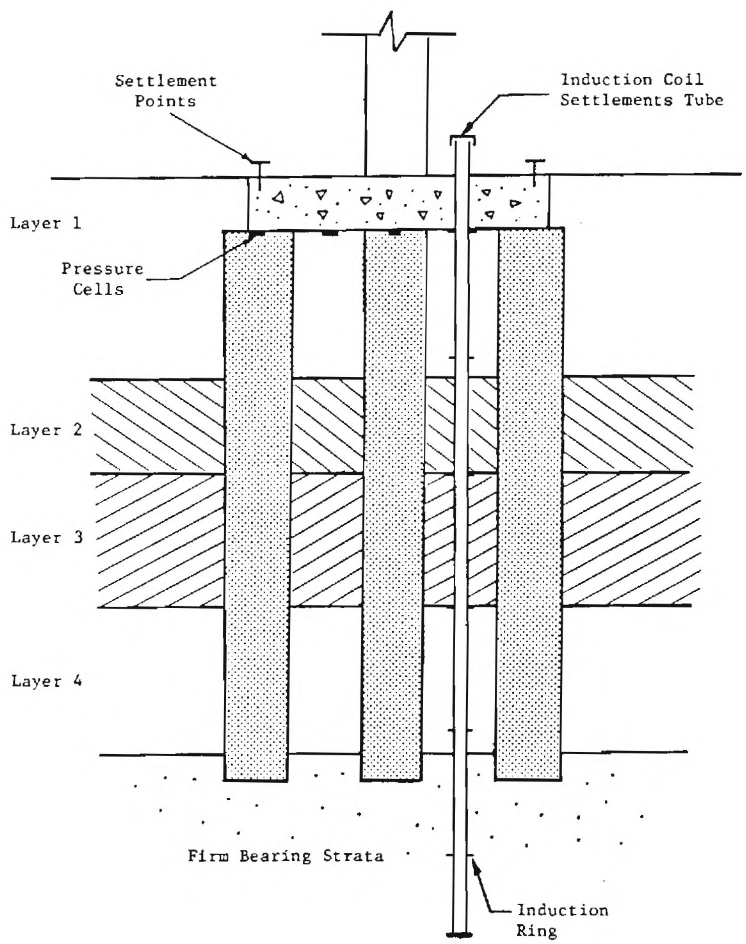


FIGURE 57. TYPICAL INSTRUMENTATION FOR FOUNDATION SUPPORTED ON STONE COLUMNS.

tional consolidation tests performed on undisturbed samples taken parallel to laminations is dependent upon the thickness of the sample, and also the thickness, orientation and spacing of laminations. Therefore, consolidation tests are not suitable for evaluating the horizontal coefficient of consolidation (or permeability) of stratified soils.

Laboratory permeability tests can, however, be used to measure both vertical and horizontal permeability. To obtain the most reliable estimate in the laboratory of the permeability ratio k_h/k_v, tests in the vertical and horizontal directions should be performed on the same specimen [99,102,104]. A cube specimen about 2.5 in. (64 mm) on a side is carefully trimmed from a tube sample. The sample is then placed in a special permeameter [104], and a constant head test is performed in one direction. The specimen is then removed from the apparatus, rotated 90°, and the test repeated to obtain the permeability in the other direction. Effects of soil reconsolidation can be determined by repeating this test sequence. The disadvantage of this test is that a special testing apparatus must be constructed. Where permeability ratios are frequently required for design the required equipment should either be constructed or purchased.

For projects where developing new equipment or modifying existing apparatus is not justified, a conventional permeameter can be used. Separate vertical and horizontal samples are tested following this approach. Because of the significant scatter in permeability results, however, a reasonably large number of samples must be tested in both directions. For a laminated, soft glacial clay, Rowe [103] found that about 20 tests were necessary to give a mean value of permeability accurate to within about 10 percent. The number of required tests would vary considerably with the soil deposit. The general recommendation is given, however, that at least 5 and preferably 10 tests be performed in each direction; the variability of results should then be analyzed statistically to determine if additional tests are required.

Field Tests. A general assessment of field methods of evaluating permeability is given in Table 5. Pumping tests are the most reliable method for evaluating the in-situ permeability, particularly for soils having anisotropic characteristics, or erratic or complex profiles [93]. Other field methods may give permeabilities less than the field value, by as much as one or two orders of magnitude. Therefore, where an accurate estimate before construction of the time rate of settlement is critical for the success of the project, field pumping tests should be conducted. The wells used to pump from should be at least 12 in. (305 mm) in diameter. Piezometers located in a line should be used to monitor the drawdown. Where groundwater flow is occurring one row of piezometers should be placed parallel and one row perpendicular to the direction of groundwater movement [93].

Where less reliable estimates of permeability are acceptable field piezometers and well point tests are often used. For ease of operation and reasonably good results, well point piezometers can be used in fine sands and silts having permeabilities greater than about 10^{-5} cm/sec. For permeabilities less than this, piezometers should be used, even in soils having more permeable laminations and seams. An excellent discussion of the advantages and disadvantages of various piezometers has been given elsewhere

TABLE 5. EVALUATION OF FIELD TEST METHODS FOR DETERMINING PERMEABILITY [100].

METHOD	TECHNIQUE	APPLICATION TO Gravel	Sand	Silt	Clay	PROBLEMS	METHOD RATING
Auger Hole	Shallow, uncased hole in unsaturated material above G.W.L	Yes	If $k>10^{-3}$ cm/sec	?	No	Difficult to maintain water levels in coarse gravels;	Poor
Test Pit	Square or rectangular test pit (equivalent to circular hole above)	Yes		?	No		Poor
Cased Borehole (No Inserts)	Falling/Rising Head, Δh in casing measured vs. time;	Yes	Yes	?	No	Borehole must be flushed; Falling Head - fines may clog base; Rising Head - liquefaction where W.L. lowered excessively	Fair
	Constant Head maintained in casing, outflow, q vs. time	Yes	Yes	?	No		
Cased Borehole (Inserts Used) (1) Sand Filter Plug	(1) Generally falling head, Δh measured vs. time only;	Yes	Yes	?	No	Single test only;	Fair
(2) Perforated/ Slotted Casing in lowest section	(2) Variable Head possible;	Yes	Yes	?	No	Cannot be used as boring is advanced;	
(3) Well point placed in hole, casing drawn back	(3) Same as for (2) above;	Yes	Yes	Yes	No		Fair to Good
Piezometers/ Permeameters (with or without casing)	(1) Suction Bellows apparatus (independent of boring); Inflow only measured vs. time	No	Yes	Yes	No	Restricted to fine sands, coarse silts; variable bellows requires $10^{-4} \leq k \leq 10^{-7}$ cm/sec.	Good (local zones)
	(2) Short Cell (Cementation); Independent of boring; Outflow ONLY measured vs. time	Yes	Yes	No	No	Carried out in adit or tunnel	
	(3) Piezometer tip pushed into soft deposits/placed in boring sealed, casing withdrawn/ pushed ahead of boring; Constant head, outflow measured vs. time; Variable Head also possible.	No	No	Yes	Yes	Possible tip smear when pushed; Δu set up in pushing tip; Danger of hydraulic fracture	
Well Pumping Test	Drawdown in central well monitored in observation wells on, at least, two 90° radial directions;	Yes	Yes	Yes (?)	No	Screened portion should cover complete stratum tested	Excellent (overall k of soil)
Test Excavation Pumping Tests	Monitoring more extensive than well pumping test, during excavation dewatering (initial construction stage)	Yes	Yes	Yes (?)	No	Expensive; Of direct benefit to contractual costing	

Note: 1. Refer to Reference 100 for additional references describing each method given in the table.

[100].

All piezometer or well holes should be either jetted, which most closely simulates the conventional construction of a stone column, or else augered. Driving should not be permitted because disturbance significantly reduces the horizontal permeability, as observed by Casagrande and Poulos [99]. After advancing, the hole should be thoroughly cleaned by flushing with water before installation of the observation system. In performing field tests air may come out of the solution if the temperature of the added water is greater than that of the groundwater. Formation of air bubbles will block the flow of water and can cause an important reduction in measured permeability.

GEOTECHNICAL REPORT

A complete geotechnical report should be prepared documenting the subsurface conditions, laboratory testing, and design phases. This report should include all field test results (test boring logs, vane shear test results, field permeability test results, etc.) and also all laboratory test results. Generalized profiles of the site should also be presented in the report.

Frequently critical dimensions, column loads, and etc. are changed several times before the final design is complete without the geotechnical engineer always being informed of these changes. To document the condition for which the design is valid, critical assumptions in the design should be clearly spelled out such as fill geometry, fill weights, construction rates, column loads, general subsurface conditions, stone column spacing, diameter, etc.

Finally, the geotechnical design engineer should maintain good communication with field personnel during the construction phase to insure that what was envisioned during design is actually achieved in the field. Frequent site inspections should also be made by the geotechnical engineer during construction. Poor communication between the designer and field personnel has frequently resulted in serious problems.

FIELD INSTRUMENTATION

Field instrumentation in stone column installations is used to monitor the construction phase to insure satisfactory performance, as well as to extend current knowledge of the behavior of stone columns for use in future designs. At the present time, only a few stone column projects have been well instrumented and the results published [27,63,71]. The Jourdan Road Terminal Project [71] undoubtedly is the most extensively instrumented, and can be used as a guide for future instrumentation. Specific instrumentation should be selected considering the (1) job requirements, (2) available personnel, (3) overall reliability and performance history of the equipment, and (4) general complexity of equipment. An excellent discussion of geotechnical instrumentation has been presented elsewhere [105].

Instrumentation for Performance Monitoring

The level of field instrumentation required to insure adequate performance where stone columns are used to improve marginal sites is dependent upon the conservatism used in the design. For a conservative design only a minimal amount of instrumentation is required. Designs using low factors of safety, which is frequently the case for embankments, require the use of more extensive field instrumentation. An important need also exists for additional field response data to verify and improve present design methods.

Bridge Pier. Fig. 57 illustrates a modest field instrumentation program that could be used to monitor the performance of a bridge pier foundation. For a bridge pier or abutment foundation, settlement is the most important variable defining performance. The settlement points placed on each of the four corners of the footing give both total settlement and tilt, giving an indication of overall footing performance. Useful settlement information is also obtained from inductance coils (or other devices) to define the magnitude of settlement in each strata, and the time rate of settlement history. Inclinometers (not shown in the figure) could be used to indicate the amount of lateral bulge occurring under the loading. Lateral bulging which can be important in soft clays and organics, is not considered in one-dimensional consolidation theory and results in larger settlements. If the weaker strata are firm and organics are not present, the inclinometers are not necessary; this in general should be the case for bridge pier foundations. Lateral spreading, however, would be more important for abutments constructed on soft to firm clays, and the use of inclinometers would give important information for this application.

Embankments. Fig. 58 shows an instrumented embankment where stone columns have been used to improve the site. For this problem, stability of the embankment is the most important consideration. The shear strength of the underlying soil increases with an increase in effective stress as the soil consolidates. Since effective stress is equal to total stress minus pore water pressure, the pore pressure and its change with time is of critical importance. Piezometers are therefore located in the vicinity of the potential critical failure circles. Note that wick drains have been used on the interior of the stone column stabilized zone to speed pore pressure dissipation in that area. Inclinometers give important information concerning the magnitude and location of lateral movement of the foundation and aid in assessing impending failure of the embankment. The inclinometers are placed just inside the edge of the toe where spreading is likely to be greatest. The inclinometers could be supplemented by toe stakes (Fig. 58) and also by "poor man's" inclinometers. "Poor man" inclinometers consist of a casing through which a probe is either lowered or pulled up from the bottom. The probe is designed so that the occurrence of important lateral movement will prevent the probe from advancing further.

Settlement plates and settlement stakes are used to monitor fill

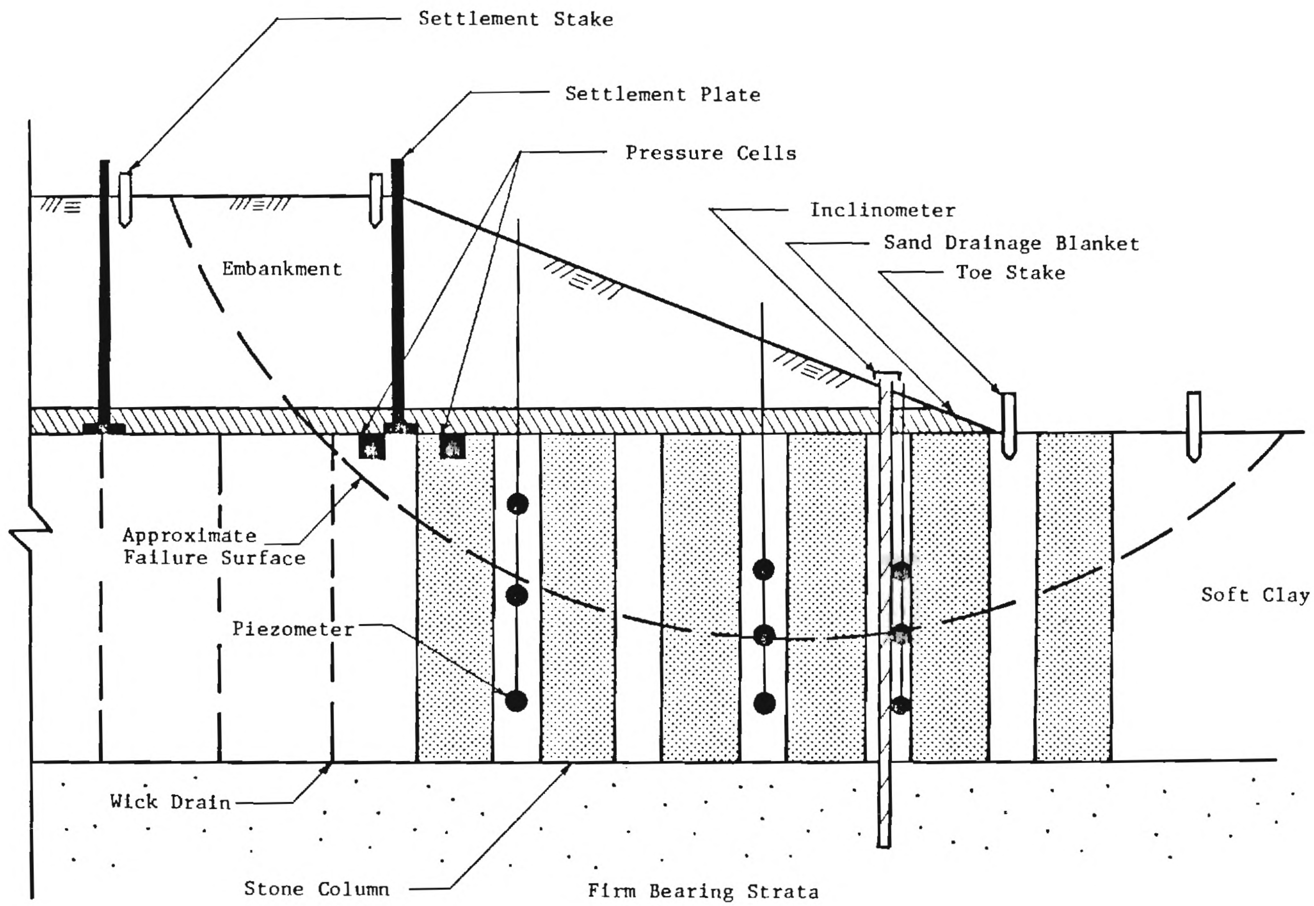


FIGURE 58. TYPICAL FIELD INSTRUMENTATION FOR EMBANKMENT CONSTRUCTED ON STONE COLUMN IMPROVED SOFT CLAY.

settlement. Pressure cells should also be placed in and between the stone columns. The information obtained concerning stress concentration would be valuable in performing a stability analysis. Placement of pressure cells at depth would also be of great use in stability analyses, but are difficult to install and may give questionable results.

As the cohesive soil consolidates, pore pressures and time rate of settlement are of primary concern. An increase in rate of settlement as a function of time indicates potential stability problems. Therefore the settlement plates and settlement stakes should be frequently monitored and a plot maintained of settlement as a function of time. Lateral spreading should also be monitored using the inclinometers and toe stakes which are simple but quite effective. Piezometers should be located between the stone columns bounding the potential failure plane location. No lateral movement theoretically occurs under the centerline of the embankment. Therefore the inclinometers should be located near the toe.

Summary. The amount of instrumentation required depends upon the subsurface conditions and the safety factor used in design. The type instrumentation used should depend upon the experience and ability of field personnel and available equipment. In general, a simple to operate piece of instrumentation which has a proven record should be selected rather than a more sophisticated instrument which would be more likely to cause problems. To obtain sufficient reliable information, duplication of instrumentation is a necessity.

SUMMARY

A thorough subsurface investigation and evaluation of geotechnical properties is essential for the design of stone columns, and the selection of the most suitable design alternative. The potential for use of stone columns and other possible design alternatives should be identified as early as possible during the subsurface investigation so that the exploration and testing program can be tailored to the specific design alternatives.

For sites underlain by very soft to firm cohesive soils, field vane shear testing is recommended. If either stone columns or densification techniques such as vibrofloation are being considered as an alternative for improving loose to firm silty sands, a sufficient number of washed grain size tests should be performed to accurately define the variation in silt content. Other special considerations for stone columns include identifying organic and peat layers, and evaluating the in-situ horizontal permeability of the compressible strata. Test pits are recommended in peat layers.

Field permeability tests should be performed where a reliable estimate of the time rate of settlement is required for the success of the project or for comparisons of different design alternatives. Field permeability tests would not, however, be required on routine projects. To minimize smear effects, well points, wells, and piezometers should be installed by jetting if the vibro-replacement method of stone column construction is to be used.

The evaluation of the permeability characteristics of a stratum is at best difficult to both perform and interpret; a high degree of accuracy should not be expected from any method. Field pumping tests give the most reliable estimate of the in-situ permeability. Laboratory permeability tests may underestimate the actual permeability under unfavorable conditions by as much as a factor of 5 to 10. Laboratory consolidation tests should not be performed to evaluate horizontal permeability.

Finally, every opportunity should be taken to instrument stone column improved ground to permit developing both improved methods of design and a better understanding of their behavior.

CHAPTER V

FIELD INSPECTION AND GUIDE SPECIFICATIONS

INTRODUCTION

Insuring proper construction of stone columns in the field is a very important but often neglected aspect. Thorough field surveillance by both the owner and contractor is essential in the construction of stone columns. Further, good communication should be maintained at all times between the inspection personnel, contractor, project engineer and the designer. This chapter considers just the construction and inspection of vibro-replacement stone columns which are the only type used to date within the United States. Further, the aspects of construction monitoring are directed towards the use of electrically powered vibrators, which have been the only type unit used in the United States to date.

STONE COLUMN INSPECTION

Stone column construction in the past has usually been considered by owners and designers a somewhat "mysterious" operation, with the inspector often having only a general idea of proper construction sequence and technique. The general construction of stone columns by the vibro-replacement and other techniques is discussed in detail in Chapter II. In this section, a summary is first given of important stone column construction/inspection aspects. This summary is followed by a detailed guide suitable for use by field personnel for the inspection of stone column construction.

Summary of Important Construction Aspects

1. Inspection records should be carefully analyzed for differences in times from one column to the next to both construct the hole and the stone column. Any significant differences may indicate (1) a change in construction technique, (2) a change in soil properties, or (3) collapse of the hole. If changes are found, determine immediately the probable cause.

2. *During construction in soft ground the probe should be left in the hole at all times and large quantities of water used to help insure (1) stability of the hole and (2) a clean stone column due to the removal of fines and organices.* An average of approximately 3,000 to 4,000 gal./hr. (11-15 m^3/hr) of water should be used during construction; more water is required during jetting of the hole, with the quantity of water decreasing as the column comes up.

3. The initial construction of a strong base at the bottom of the stone column is important to insure proper performance. Therefore, additional penetrations of the probe are desirable together with extra care in construction during compaction of the first several increments of stone column backfill. When stone is first dumped down the hole some of it will probably penetrate into the soft clay surrounding the hole near the surface. Therefore, the diameter of the column at the base will not be as large as calculations indicate.

4. *The occurrence of unexpected peat layers should be brought to the immediate attention of the project engineer and the designer.* The presence of peat layers has been found to cause problems in the performance and construction of stone columns. As a rule-of-thumb, the thickness of a peat layer should be no greater than the diameter of the column. If a peat layer is encountered of thickness greater than the stone column diameter, two probes can be fastened together to construct a large diameter stone column.

5. If organics such as peat are encountered caution should be exercised to flush this material out of the hole; extra flushings are necessary to assure proper removal of the peat. These extra flushings may enlarge the diameter of the hole in the peat and increase the stone take in this area. The stone column should be built as rapidly as possible in peat, silts and sensitive soils.

6. If localized areas of very soft soils are encountered, it may be desirable to use a coarse gradation such as Alternate No. 2 given in the Guide Specifications if rapid construction does not solve the problem.

7. Stone may "hang up" in the hole before it gets to the bottom. To prevent this and to clean out any soil which may have been knocked loose, the probe should be lifted and dropped (stroked) 6 to 10 ft. (2 to 3 m) several times after the stone has been added. Note: If the hole collapses while the probe has been lifted, the probe will not return to the correct depth. Also the probe should not be lifted completely out of the hole during stroking.

8. When the power consumed by the vibrator motor reaches the specified value, this primarily means that good contact exists between the probe and the stone. Reaching the specified power consumption alone is therefore not a complete guarantee construction is satisfactory and a high density has been achieved; it does not eliminate the need for carefully watching the entire construction sequence. Power consumption as defined by ammeter reading is, however, a useful field control that can be continuously monitored. Also it tends to keep the operator alert and encourages him to do a conscientious job.

9. In constructing stone columns in sand getting the required ampere draw on the motor is usually no problem; in soft clays it can be. The crane operator can build up misleadingly large amp readings by dumping excessive quantities of stone into the hole, and then quickly dropping the probe. Such a practice should not be permitted.

10. In general larger horsepower vibrators require more amps either in

the unloaded (free standing) position or loaded as they construct a stone column. For example, one 175 horsepower vibrator draws 130 amps in the unloaded condition. Obviously a specification requirement of, say, 80 amps which has been often used in the past has no meaning for the very large horsepower vibrators now coming into use. Therefore, the recommendation is given subsequently in the specifications section to use as a minimum the free standing amp reading plus at least 40 additional amps during construction of the column. Further, a total amp reading of less than 80 should probably not be permitted to insure minimum equipment capability.

11. *As an important supplement to the ammeter reading, carefully watch the amount of repenetration of the probe after stone has been added to the hole. The first repenetration should extend through the newly placed stone, with less penetration occurring on successive repenetrations.* Some engineers feel good repenetration is even more important than the ammeter reading.

<u>Inspection Guidelines for Stone Column Construction</u>

A discussion of critical terms in inspecting stone columns was given in the preceding section. The following checklist serves as a general guide for inspection personnel to systematically monitor stone column construction.

Construction of stone columns requires special equipment and technical expertise. Construction of stone columns should only be undertaken by contractors experienced in this type work.

I. VIBRO-REPLACEMENT INSTALLATION EQUIPMENT:

The following items are to be checked or noted:

1. Type of vibro-replacement equipment as specified in contract
2. Vibrator Characteristics

    a. Diameter of vibrator barrel (in./mm)
    b. Diameter of vibrator including stabilizing fins (in./mm)
    c. Length of vibrator and follower tubes (ft./m)
    d. Horsepower
    e. Amplitude of free vibration (mm)
    f. Frequency of vibration (rpm)
    g. Eccentric moment
    h. Jets

        (i) Number and location of jets
        (ii) Inside diameter of jets

3. Water Supply to Vibrator

    a. Pump type and capacity
    b. Supply line type and inside diameter
    c. General condition of water supply line (condition of hoses, leaks, constrictions, etc.)
    d. Quantity of water used per hour

e. Operating pressure

II. CRUSHED STONE:

The following items are to be periodically checked as provided for in the specifications or as considered necessary:

1. Contamination of the stone as it comes from the supplier including weak aggregate, sand, organics, or other deleterious materials.
2. Gradation of the stone and other applicable requirements as set forth in the specifications.
3. General contamination of the stone due to the method of stock-piling and moving it on site.

III. SAND WORKING PLATFORM:

The following items are to be periodically checked as provided for in the specifications or as considered necessary in the field:

1. Sand working platform thickness
2. Gradation of sand
3. Construction of the platform should be conducted so as to cause a minimum amount of disturbance to the underlying soils. For example, the working platform should be constructed by pushing the sand out onto the soft soil from the completed platform using light equipment.
4. If a geotextile is required below the sand blanket it should meet specifications including material type (nylon, polyester, poly-propylene, polyethylene, etc.), manufacturing process (woven, nonwoven, heat bonded, needle punched, etc.), material weight and strength.

IV. CALIBRATION FOR QUANTITY OF STONE:

To permit estimating the in-situ diameter of the stone column after construction the following data is required:

1. Determine the maximum and minimum density of the stone following ASTM Method C29 before stone column construction begins.
2. Determine the volume of the bucket to be used to place the aggre-gate in the jetted stone column hole. The bucket volume can be determined from the manufacturers' literature or by filling it with a known quantity of water or loose aggregate.

V. STONE COLUMN INSTALLATION:

The following items should be checked or noted during the installation of each stone column:

1. Record the stone column number, and the date and time installation begins.
2. Record the time required to form the hole.
3. Record the stone column length and bottom tip elevation.
4. Observe after jetting that the hole is properly flushed out before the stone is placed. The hole is flushed out by raising and dropping the vibrator at least 10 ft. (3 m) as provided in the specifications.
5. Observe that the vibrator is left in the hole during placement of the stone.
6. Observe during stone placement that a good upward flow (3000-4000 gph, or 11-15 m^3/hr average) of water is maintained at all times to avoid possible collapse of the hole. The upward flow is provided by keeping the jets running on the sides of the vibrator.
7. Observe that after the stone is dumped down the hole the vibrator is lifted and dropped (stroked) a short distance (6 to 10 ft., or 2-3 m) several times to insure the stone reaches the bottom and does not arch across the hole; the vibrator should not be completely removed from the hole during stroking.
8. Estimate the lift thickness placed being sure it conforms with specifications.
9. Observe that the vibrator goes through the recently placed lift of stone during the first penetration; additional repenetrations should have smaller penetration depths into the lift.
10. The specified reading on the ammeter should be developed during the construction of each lift. A continuous record of the ammeter reading may be made by the contractor. This record should be periodically checked to be sure the equipment operator is satisfying the ammeter specification.
11. Record the total number of buckets of stone required to construct each stone column. Also, keeping a record of the number of buckets placed in selected lengths of column (and hence the quantity of stone used per unit length) permits estimating the approximate diameter of the stone column as a function of depth. Determining the variation of stone column diameter with depth is desirable to obtain an indication of possible problem strata and the physical mechanics of the construction process. Therefore, for most jobs the detailed records necessary to define the variation of diameter with depth should be kept during installation of at least the first few stone columns and for selected columns thereafter. If problems are anticipated during installation of subsequent columns, detailed records of stone consumption should be kept for each stone column.
12. Record the total time required to construct each stone column.
13. Carefully observe each stone column after construction and measure the diameter. (Note: Because of low overburden pressure and erosion, the diameter at the surface is generally larger than the average diameter.)
14. Note any unusual phenomenon during or after construction; for example, the subsidence of a stone column, excessive times required to form the hole or construct the stone column, or the presence of undergound obstructions. The occurrence of any of

these problems or other unusual events should be immediately called to the attention of the project engineer.

15. Note the technique, equipment and adequacy of the method used to penetrate any obstructions.
16. Call the presence of natural gas or unusual odors to the immediate attention of the project engineer and the contractor.
17. Record general comments concerning the adequacy of the overall construction process including flushing the hole, keeping the probe in the hole during stone placement and maintaining upward flow of water, repenetrating the stone and achieving the specified ammeter reading. Any continuing problems should be brought to the attention of the project engineer and the designer.

VI. ENVIRONMENTAL CONSIDERATIONS

Periodically inspect the site to insure the plans and specifications are being met with regard to all environmental requirements and restrictions including any siltation ponds, straw or fabric silt barriers, and general disposal of the effluent from the construction project. Immediately inform the project engineer of any problems with meeting environmental site requirements.

VII. GENERAL RECORDS

The inspector should keep up to date the following records:

1. A table summarizing the project status including: stone column number, date of construction, stone column length, average diameter, diameter at the surface, total quantity of stone used, total construction time, time to jet hole, and time to place and densify stone column.
2. A plan of the stone columns showing as a minimum the location and number of each stone column, date completed, total quantity of stone used and total construction time. Each completed stone column should be colored in red on the drawing.
3. Maintain a record on a weekly basis indicating the general adequacy of the environmental controls and construction progress of the project. Also, periodically take photographs for a permanent record of the site showing the condition of the site with respect to environmental considerations, equipment, and any special features.

STONE COLUMN SPECIFICATIONS

A review was made of a number of specifications used on past projects for the construction of stone columns [106]. Specifications of this type can be written to follow either of the following two extremes or can be somewhere in between:

1. Detailed specifications completely defining each step of the construction process such as the Alaska Specifications or the Kavala Specifications [106].

2. End result specifications which require the Specialty contractor to improve the site to, for example, support a certain design bearing pressure or not exceed a specified settlement; the Vancouver Specifications [106] are an example of this extreme.

Unless trial stone columns have been constructed beforehand, giving too much detail in the specifications is probably not the best approach in most instances. Under these conditions the Specialty contractor should have some latitude in the equipment used, and details of the construction operation. On the other hand, for ground improvement projects utilizing stone columns designed by the owner or his representative, specifying an end result, considering the many uncertainties associated with stone column construction, would not be appropriate either. The specifications given are intended as a general guide for stone column projects where end result specifications are not used. These specifications indicate generally accepted construction practices. *The guide specifications should be modified as necessary to meet the special requirements of each project and the philosophy of the designer.* Only qualified Specialist contractors should be selected to perform stone column work.

GUIDE SPECIFICATIONS FOR STONE COLUMNS

A. GENERAL

Ground improvement shall be performed by constructing stone columns formed by deep vibratory compaction using imported crushed stone (or gravel). The principal items of work included in these specifications are[1]:

1. Construction of stone columns, complete in-place including layout.
2. Furnishing crushed stone (or gravel) as required for the stone columns.
3. Furnishing equipment, electrical power, water and any other necessary items for stone column installation.
4. Control and disposal of surface water resulting from stone column construction operations.
5. Construction of sand (or stone) working platform and necessary access to site (this may be included under another contract).
6. Construction and removal of silt settling ponds or similar facilities as required, and the regrading of the site as required.
7. Stockpiling and disposal of silt from the site if necessary.
8. Load testing of stone columns as specified.

The installation of all stone columns under the contract shall be the responsibility of one Specialist contractor. No part of the contract may

[1]Site clearing and grading are not included.

sub-let without prior approval of the Engineer[1]. The Specialist contractor shall furnish all supervision, labor, equipment, materials and related engineering services necessary to perform all subsurface ground improvement work.

The Specialist contractor shall state in his bid the type and number of vibroflots and his general method of operation including construction schedule.

B. REQUIREMENTS OF REGULATORY AGENCIES

Prevention of Nuisance. The Specialist contractor shall comply with all laws, ordinances, and other regulatory requirements governing the work including those pertaining to the prevention of nuisance to the public and adjoining property owners by noise, impact, vibration, dust, dirt, water, and other causes. The contractor shall immediately discontinue any construction or transportation method that creates any such nuisance, and perform the work by suitable lawful methods at no extra cost to the owner.

Disposal of Water. The Specialist contractor shall (1) meet all applicable laws and regulations concerning surface runoff, siltation, pollution and general disposal of the effuent from the construction of the stone columns and general site work. (2) Construct and relocate temporary ditches, swales, banks, dams, and similar facilities as necessary to control the flow of surface water during the work. Remove them when no longer required, and regrade the affected areas for acceptable drainage as specified for site grading. (3) Construct silt settling ponds as required in locations indicated or approved. Ensure that earth banks and water control devices are safely designed and prevent inadvertent discharge into watercourses off the site. Stockpile and dispose of all silt as approved by the Engineer. (4) Remove settling ponds and other structures when no longer required and regrade the areas for acceptable drainage as specified for site grading.

C. MATERIALS

The Specialist contractor shall notify the Engineer in writing of proposed sources for rock and sand at least 14 days before importation operations begin. This material will be sampled at the source and tested by the Owner/Engineer to determine compliance with the requirements specified. The rock and sand shall be brought to the site only after receiving written authorization from the Owner.

Stone. The crushed stone (gravel) for column backfill shall be clean, hard, unweathered stone free from organics, trash, or other deleterious materials. When subjected to the magnesium sulfate soundness test (ASTM C88), the percent weight loss shall be not more than 15 percent. When tested according to ASTM C131 the crushed stone (gravel) shall have maximum loss of 45 percent at 5000 revolutions. The gradation shall conform to

[1]The Engineer is used throughout the specifications to indicate the designated representative of the owner.

the following for the vibro-replacement process[1]:

Sieve Size (ins.)	Alternate No. 1 Percent Passing	Alternate No. 2 Percent Passing	Alternate No. 3 Percent Passing	Alternate No. 4 Percent Passing
4	-	-	100	-
3.5	-	-	90-100	-
3.0	90-100	-	-	-
2.5	-	-	25-100	100
2.0	40-90	100	-	65-100
1.5	-	-	0-60	-
1.0	-	2	-	20-100
0.75	0-10	-	0-10	10-55
0.50	0-5	-	0-5	0-5

The Owner shall furnish laboratory test results obtained by him or his designated representative for the following tests:

(a) Gradation in accordance with AASHTO T-27.
(b) Specific Gravity in accordance with ASTM C127
(c) Density of loose stone in accordance with ASTM C29.
(d) Density of compacted stone in accordance with ASTM C29.

A new series of tests may be performed for each 2000 tons, or as required by the Engineer, of stone or sand furnished from each source.

Sand. The sand used for the working platform shall be hard, natural or manufactured sand free from organics, trash or other deleterious materials. The sand shall be well-graded, contain less than 15 percent passing the Number 200 sieve, and have a mean diameter of at least 0.2 mm.

Approval of Stone and Sand. Both the crushed stone (gravel) and sand source shall be approved in writing by the Engineer before it is imported to the site.

Water. Fresh, brackish, or sea water or any combination, free of all substances deleterious to the work may be used.

[1]In general, Alternate No. 1 or No. 2 gradation is recommended. For very soft organic zones, Alternate No. 2 and rapid construction should be tried; if this does not work use Alternate No. 3. Alternate No. 2 or No. 4 can be used if a large topsize aggregate is not available. A specific gradation should be selected by the Owner and written into the specifications based on site conditions and available stone gradations.

D. EQUIPMENT AND METHODS

At the beginning of the project[1], test stone columns shall be installed at locations designated by the Engineer, for the purpose of establishing quality control procedures.

Vibrator. Stone columns shall be installed by jetting, using vibratory probes 14 to 19 in. (360-480 mm) in diameter (not including the fins). The vibrator shall have an eccentric mass located in the lower part of the probe which shall be capable of developing the required vibration characteristics at a frequency of 1600 to 3000 rpm. The vibrator shall be driven by a motor having at least a 60 hp[2] rating that is capable of developing a minimum centrifugal force, in starting, of 15 tons gyrating about a vertical axis. The minimum double amplitude (peak to peak measurement) of the probe tip shall be not less than ten (10) mm in the horizontal direction when the probe is in a free suspended position. Note: These rather general requirements on the vibrator are satisfied by most available probes; field tests are needed to define the best vibrator for stone column construction.

Installation. The construction technique and probe shall be capable of producing and/or complying with the following:

1. Produce approximately circular holes.

2. The probe and follower tubes shall be of sufficient length to reach the elevations shown on the plans. The probe, used in combination with the flow rate and available pressure to the tip jet, shall be capable of penetrating to the required tip elevation. Preboring of stiff lenses, layers or strata is permitted.

3. The probe shall have visible external markings at one (1) foot increments to enable measurement of penetration and repenetration depths.

4. Provide for supplying to the tip of the probe a sufficient quantity of washwater to widen the probe hole to a diameter at least 12 in. (305 mm) greater than the probe to allow adequate space for stone backfill placement around the probe. The flow of water from the bottom jet shall be maintained at all times during backfilling to prevent caving or collapse of the hole and to form a clean stone column. An average flow of 3000 to 4000 gph (11-15 m^3/hr) of water shall be maintained throughout construction. The flow rate will generally be greater as the hole is jetted in, and decrease as the stone column comes up.

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[1]Refer to Chapter VII for a discussion of load testing.

[2]Several competent contractors believe that for stone column construction in weak soils the horsepower, centrifugal force, and vibration amplitude are less important than in the densification of sand. They feel more relaxed specifications can therefore be used for stone column construction than for sand densification.

5. After forming the hole, the vibrator shall be lifted up a minimum of 10 ft. (3 m), dropped at least twice to flush the hole out. The probe shall not, however, be completely removed from the hole.

6. Form the column by adding stone to fill the hole in 24 to 48 in. (0.61-1.22 m) lifts. Compact the stone aggregate in each lift by repenetrating it at least twice with the horizontally vibrating probe so as to densify and force the stone radially into the surrounding in-situ soil. The stone in each increment shall be repenetrated a sufficient number of times to develop a minimum ammeter reading on the motor of at least 40 amps more than the free-standing (unloaded) ampere draw on the motor[1], but no less than 80 amps total.

7. Stone columns shall be installed so that each completed column will be continuous throughout its length.

During construction, if the stone columns are consistently over or under the average effective diameter[2] of __ feet, as defined in Section E, and the workmanship and material have been consistent with those used in previously acceptable work, this may indicate that the soil conditions have changed from those encountered during the earlier work. The Contractor shall cease operations in the immediate area of work and notify the Engineer. The Engineer will make a determination of whether it is necessary and the extent to which it is necessary, to adjust the pattern and spacing.

Erosion of Working Platform. If erosion of upper granular working platform material occurs, the depressions shall be backfilled with sand which meets the specification for the working platform. Such backfilling shall be at the Contractor's expense.

The working surface shall be cleaned at the completion of the stone column construction of all unsuitable materials washed up from the stone column holes. Such unsuitable materials include clay or silt lumps, wood fragments or other organic matter. If, in the opinion of the Engineer, these materials create "soft spots" or zones of compressibility or weakness in connection with the placement of overlying embankment materials, these unsuitable materials shall be disposed of in a manner approved by the Engineer.

Workmanship. The Engineer's determination of the quality and adequacy of workmanship employed in installation of the stone columns in the various areas will include consideration of the Contractor's consistent use of the same procedures, methods, and construction performance rates as those used in installing initially acceptable stone columns.

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[1]Refer to the section on Stone Column Inspection in this chapter for a discussion of the limitations of using ampere reading to control construction.

[2]The diameter of the constructed stone column varies with many factors including construction equipment, technique and also the site conditions; refer to Chapter VII.

E. TOLERANCES

Location. No vibration center or stone column shall be more than 4 in. (100 mm) (10 in. or 250 mm for embankment stabilization work) off its correct center location at the working platform level as shown on the approved plans, except as specified in Section F. The axis of the stone column shall not be inclined from the vertical by more than 2 in. in 10 ft. (50 mm in 3 m) as indicated by the tilt of vibrator and follower tubes.

For any group of 50 consecutively installed stone columns, the average diameter over its length shall not be less than __ feet, and not more than one stone column in this group shall have an average effective diameter over its length of less than 90 percent of the average diameter for the group. If the columns do not meet the above requirements then the installation operation must be adjusted to produce the specified diameters or, if approved by the Engineer, the stone column spacing decreased at the Specialists contractor's expense to give the same percentage of area improved with stone columns.

During construction, if the stone columns are consistently over or under an average effective diameter of __ feet and the workmanship and material have been consistent with those used in previously acceptable work, the Engineer may direct to change the operation as the soil conditions may have changed.

The average effective stone column diameter shall be calculated using the inplace density of the stone and the weight of stone used to fill the hole. For calculation of constructed column diameter, the inplace density shall be assumed to be equal to 80 percent of the relative density determined by using the loose and compacted densities of the stone as specified in Section C.[1] The weight of stone required to construct the stone column shall be based on the equivalent number of full buckets dumped down the hole and the loose stone density determined in Section C.

F. OBSTRUCTIONS

A 15 in. (380 mm) maximum horizontal deviation from indicated column location will be allowed without prior authorization from the Engineer when an obstruction is encountered; the presence of any obstruction shall be reported to the Engineer and described in the records. When a deviation greater than 15 in. (380 mm) is caused by an obstruction, the contractor shall stop work, move to another compaction point and immediately notify the Engineer. The Engineer may at his option authorize one or several of the following: (1) position the compaction point a short distance away from the original position, (2) additional compaction points to bridge the obstruction, (3) remove the obstruction, replace removed soils, and again jet the column hole in the indicated location or (4) perform other removal or relocation operations. The owner will pay the Contractor for authorized

[1]A better approach would be to use the measured inplace density of the stone column. At the present time data is not available on the variation of density with depth within the stone column.

work to remove obstructions or for performing directed relocation operations, except shifting the compaction point, based on accepted contract unit prices.

G. CONSTRUCTION RECORDS

The contractor shall provide competent and qualified personnel to continuously observe and furnish to the Engineer recorded logs of the following data to be obtained during column installation:

1. Stone column reference number.

2. Elevation of top and bottom of each stone column.

3. Number of buckets of stone backfill in each stone column.

4. Vibrator power consumption during penetration of vibrator, and vibrator power consumption during compaction of stone column. The date and column identification shall be written on each record. Note: A continuous graphical record is desirable of the amperage draw of the vibroflot motor during the construction of each stone column. Such records should be maintained where more than one vibrator is to be used with a single inspector, or where one vibrator is used without full-time inspection.

5. Time to penetrate and time to form each stone column.

6. Details of obstructions, delays and any unusual ground conditions.

The Owner shall furnish a full-time inspector to observe stone column construction.

The recorded logs of the above information signed by the Specialist contractor's representative and the Owner's inspector shall be submitted to the Engineer each week.

The stationing, top elevation, limits, pattern, spacing and approximate depths for the stone column work are shown on the plans. The Contractor shall prepare construction drawings showing specific stone column locations, identification number, and estimated depth of compaction points. These drawings are to be submitted to the Engineer for approval in accordance with contract requirements. During progress of work these drawings are to be annotated to show the compaction points completed each day.

At the end of the ground treatment work, a report shall be prepared by the Specialist contractor and submitted to the Owner giving details of the plant and methods used, production rates, and the performance of the site during treatment, together with all load test results and calculations based on the data obtained during the stone column construction.

H. METHOD OF MEASUREMENT

The accepted quantity of stone columns, including test columns, will be measured in total linear feet of all columns complete inplace. Measurement will be from the bottom of each column to the elevation given on the drawings. Measurement of each column will be to the nearest one foot (300 mm).

I. BASIS OF PAYMENT

The contractor will be paid a lump sum amount for set-up and removal cost for mobilization of facilities and equipment for stone column production. In addition, stone columns will be paid for at the contract unit price per linear foot. A unit price should be given for each possible stone column spacing. The above payments shall constitute full compensation for development of stone column holes; for furnishing and placing aggregate; for providing records, logs, and reports; and for providing all labor, supervision, tools, equipment, materials and incidentals necessary to complete the work. Load tests shall be conducted on a lump sum basis for each test as specified by the engineer. Note: The sand blanket working platform material and placement is normally a separate pay item.

SHORT-TERM LOAD TESTS

Load tests will not be required on all stone column projects. A guide specification is given in this section describing a vertical, short-term (undrained) load test. Where settlement is of primary concern, a long-term (drained) vertical load test is required. Both vertical and direct shear load tests are discussed in Chapter VII.

The contractor shall furnish all required concrete slabs, weights, equipment, gages, and instrumentation for the tests. The test method shall be in accordance with the following:

1. Definition

A preliminary stone column shall be a stone column installed prior to the construction of the working stone columns to establish that the system the Specialist contractor proposes to use and the proposed centers of the stone columns satisfy the requirements of the specifications. A non-working stone column shall be a stone column installed during the period of the installation of the working stone columns to verify the predicted capacity of a working stone column. The preliminary and non-working stone columns shall be of the same dimensions and materials, and constructed with the same plant and in the same manner as the working stone columns. The dimensions and lengths of individual preliminary stone columns and non-working stone columns shall be as approved by the Engineer. Preliminary and non-working stone columns shall be paid for as specified in the contract.
Note: Depending upon the project, load tests may be performed upon the working stone columns.

2. Test Program

The specified contract rates shall include supplying all necessary labor, materials, plant, and equipment necessary (1) to construct the stone columns, (2) to apply the test load, and (3) to measure the deflection under load, all in the prescribed manner. Details for conducting the tests as described in the specifications shall be submitted for approval by the Engineer before installation of the test stone columns.

3. Equipment

a. Capacity of Load Test Equipment. The test equipment shall be capable of safe application of three times the calculated working load for preliminary tests on non-working individual stone columns, and one and a half times the calculated working load in the case of individual stone columns required for the work.

b. Reactions for Load Tests

(1) Deadweight. If deadweight is used to provide the reaction for the hydraulic jack, it shall be supported on a suitable platform to allow safe access to the loading and measuring equipment at all times. The nearest edge of the platform supports shall be at least 10 ft. (3 m) from the periphery of the stone column.

(2) Reaction Piles. If tension piles are used to provide the reaction for the hydraulic jack, these piles shall not be closer to the stone column than 10 ft. (3 m). Under-reamed tension piles will not be permitted.

c. Load Measurement. The test load shall be applied vertical and concentric to the stone column by means of a hydraulic jack with a pump of capacity meeting test requirements. The applied load shall be measured by an approved load cell or proving ring calibrated in divisions not exceeding 2 percent of the maximum load to be applied. A certificate of calibration for the load cell or proving ring, obtained within one month prior to the test, shall be provided.

d. Deflection Measurement. Observations of vertical deflection of the head of the stone column shall be made with a minimum number of three dial gages having a 2 in. (50 mm) travel and graduated in 0.001 in. (0.025 mm) divisions. The tips of the stems of the dial gages shall rest on machined metal or glass securely bedded on the head of the concrete load footing.

Metal blocks 1 in. (25 mm) thick ±0.001 in. (±0.025 mm) with surface ground, parallel surfaces shall be provided to obtain continuity in extending the range of the gages. Two of the dial gages shall be positioned diametrically opposite each other, at equal distances from the center of the stone column; the third shall be at right angles to the other two near the edge of the footing.

The readings shall be referenced to two rigid steel beams the ends of which shall be fixed to reliable steel supports. The supports shall penetrate not less than 10 ft. (3 m) below the ground surface, and shall be located not closer than 10 ft. (3 m) from the center of the test stone column, away from the influence of the reaction system.

The elevation of the steel supports of the reference beams and the deflection of the stone column shall be verified with a precise surveyor's level with reference to a permanent benchmark. The leveling instrument and level rod shall be capable of providing direct readings to an accuracy of 0.001 ft. (0.30 mm).

e. Protection of Measuring Equipment. The measuring equipment shall be protected throughout the period of the test against adverse effects of rain, sun, frost, vibration, and other disturbances that may affect its reliability. Temperature readings shall be taken at maximum intervals of one hour throughout the test period.

4. Application of Load

The __________[1] load tests shall be located as shown on the plans. A rigid, reinforced concrete foundation(s) shall be placed over the stone column(s) having the shape(s), dimensions, and location designated on the plans.

a. First Load Application - Working Stone Column (Maintained Load Test). The test load shall be applied to the stone column in increments equivalent to 20 percent of the calculated working load until the working load is attained. Each load increment shall be maintained for 15 minutes before the next increment is added. The calculated working load shall be maintained for a minimum of 12 hours thereafter and/or until the rate of settlement does not exceed 0.002 in. (0.05 mm) per hour.

Unloading shall then take place in five equal decrements with each intermediate load being maintained for a minimum period of fifteen minutes.

Zero load, at the end of the cycle of unloading, shall be maintained for a minimum of four hours and/or until the rate of recovery does not exceed 0.002 in. (0.15 mm) per hour.

The elevation of the rigid steel beam supports shall be verified by precise surveyor's level with reference to the permanent bench-mark before the commencement of the load test and at zero

[1]The number and type (preliminary, non-working or working stone column) of load tests shall be given in the specifications.

load at the termination of the test.

b. Second Load Application - Working Stone Column (Maintained Load Test). In the second load cycle the same load increments as before shall be applied to a maximum load equivalent to 1.5 times the calculated working load. Each load increment shall be maintained for 15 minutes before adding the next increment. The maximum load shall be maintained for a minimum of 12 hours thereafter and/or until the rate of settlement does not exceed 0.002 in. (0.05 mm) per hour.

Unloading shall then take place in six approximately equal decrements with each intermediate unloading decrement being maintained for a minimum period of fifteen minutes. Zero load, at the end of the cycle of unloading, shall be maintained for a minimum of four hours, and/or until the rate of recovery does not exceed 0.002 in. (0.05 mm) per hour.

The elevation of the rigid steel beam supports shall be verified by precise surveyor's level with reference to the permanent benchmark before the commencement of the load test and at zero load at the termination of the test.

c. Non-Working Stone Column and Preliminary Test Stone Columns. On completion of the maintained load test (first load application) on a non-working or a preliminary stone column, each load increment shall be maintained for 15 minutes before the next increment is added. The same load increment as in (a) above shall be used. Stone column settlement shall be measured at each increment, with the test being continued until failure or the specified load is attained.

Unloading shall be in at least five approximately equal decrements. Each unloading decrement shall be maintained for a minimum of fifteen minutes. The elevation of the rigid beam supports shall be verified before the commencement of the load test and at zero load at the termination of the test.

5. Notification, Supervision, Reports

The period between the construction of a stone column and the commencement of the application of the test loads shall be at least 24 hours. The contractor shall give at least 48 hours notice of the commencement of each load test to the Engineer.

The Contractor shall keep the test under continuous and competent supervision to the satisfaction of the Engineer. All necessary facilities shall be provided to enable the Engineer to verify readings during the progress of the test. The Contractor shall send to the Engineer within one week of the completion of each test four copies of all records and results in graphical form. This information shall include a load deflection curve plotted to scales so as to approximately fill a standard size page.

SUMMARY

The construction phase of ground improvement work using stone columns is even more important than for conventional foundations. Therefore a competent contractor is necessary who is paid a fair price for his work. Also, full-time inspection by a qualified engineer, geologist or senior technician is very important. Finally good communication should be maintained between the inspector, project engineer, designer, and contractor. Periodically the designer should inspect the project whether problems have been encountered or not.

CHAPTER VI

SELECTED STONE COLUMN CASE HISTORIES

INTRODUCTION

Five case histories are given in this chapter to illustrate selected applications of stone columns. The first two case histories show how stone columns were used together with a Reinforced Earth retaining wall. The third case history illustrates the use of stone columns to support an embankment. The fourth case history shows how stone columns were used to improve both the resistance to liquefaction and the ability of the soil to carry foundation loads. Finally, an application of rigid stone columns is described at a site where conventional stone columns cannot be used due to the presence of a peat layer at the surface.

HIGHWAY FILL/REINFORCED EARTH WALL

Clark Fork Highway runs along the edge of Lake Pend Oreille in Idaho. Because of the presence of loose sandy silt lake deposits that dip steeply towards the lake (Fig. 59), a conventional embankment fill was not feasible due to an inadequate safety factor with respect to sliding [10]. The sandy silt present at the site had an average measured shear strength of about 300 psf (14 kN/m^2) with an angle of internal friction of 29°. For stability analyses the sandy silt was assumed to have a shear strength of 150 psf (7 kN/m^2) and 23°. This reduced shear strength was used because of the high frequency of sample loss, and low standard penetration resistances encountered during the subsurface investigation.

Earlier, during construction of another portion of the embankment, 30,000 yd^3 (23,000 m^3) of material slid into the lake. Therefore, stability of this final segment of the embankment was of great concern. A conventional embankment (without Reinforced Earth or stone columns) had a calculated safety factor of 0.9 to 1.22 with respect to a stability failure. Use of Reinforced Earth (without stone columns) permitted a vertical face on the lake side which greatly reduced the weight of the fill. As a result the safety factor increased to between 1.25 and 1.4. Supporting the Reinforced Earth embankment on stone columns further increased the safety factor to 1.36 at the most critical section; this level of improvement was considered acceptable.

The final design consisted of a 25 ft. (7.6 m) fill and Reinforced Earth wall supported on stone column improved ground as shown in Fig. 59. On this project 851 stone columns were constructed on a 7.0 ft. (2 m), equilateral triangular grid. The average stone column length was 42 ft. (12.8 m), and

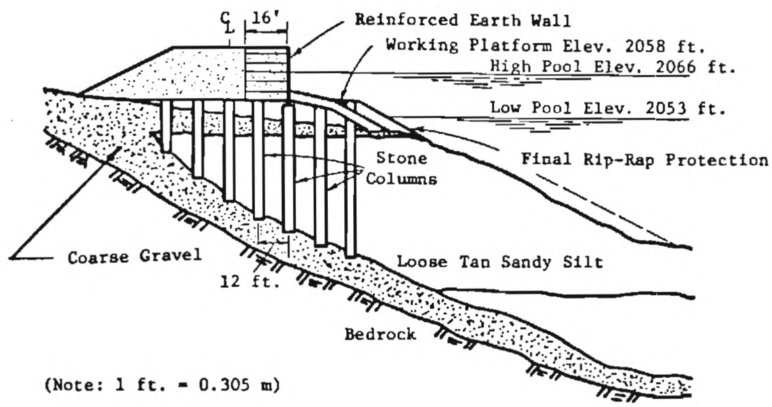

FIGURE 59. TYPICAL PROFILE OF STONE COLUMNS AND 25 FT. HIGH REINFORCED EARTH RETAINING WALL - CLARK FORK HIGHWAY [10].

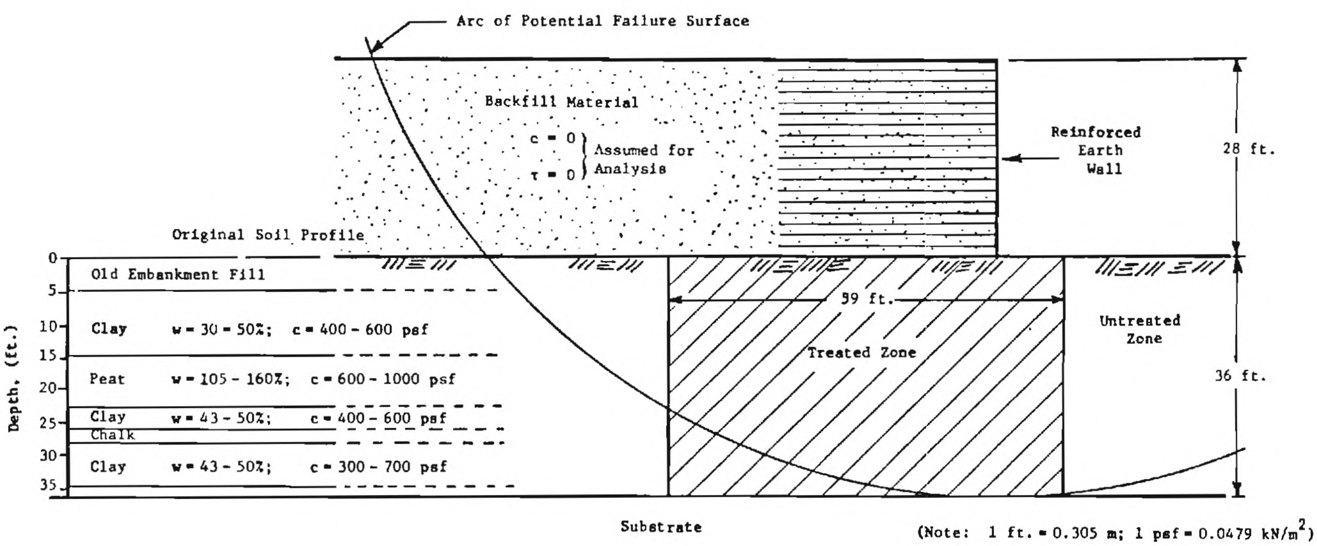

FIGURE 60. SUBSURFACE CONDITIONS AND REINFORCED EARTH WALL AT ROUEN, FRANCE [63].

the average diameter was estimated from the volume of stone used to be about 3.3 ft. (1 m). The gradation of the stone used in the columns is given in Table 6. A 5 ft. (1.5 m) free board above low pool level in the lake was provided by constructing a granular blanket working platform. The safety factor of the working platform was only 1.1 in the critical section. Therefore, the platform and stone columns were constructed in 50 ft. (15 m) segments. The stone for the platform was end dumped from the shoreline towards the lake, with stone column construction being carried out in the same direction.

The project was completed in 27 working days using four rigs. Inclinometers and piezometers were installed and monitored to verify stability during and after construction. Approximately 20 percent of the loose sandy silt was replaced by stone in the stabilized zone (i.e., $a_s = 0.20$). The total inplace cost in 1975 of the 35,638 lineal ft. (10,870 m) of stone columns required to stabilize the sandy silt was $8.10/ft. ($26.60/m). Another design alternative was to support the roadway on a bridge structure. The bridge structure, however, was estimated to cost a little more than two times the Reinforced Earth-stone column support scheme used.

APPROACH FILL/REINFORCED EARTH ABUTMENT

A 28 ft. (8.5 m) high approach fill and Reinforced Earth abutment wall was constructed over a soft clay improved using stone columns along the River Seine at Rouen, France [63]. The site consisted of about 36 ft. (11 m) of soft clay having an 8 ft. (4.5 m) peat layer sandwiched within it at a depth of 15 ft. (4.5 m) as illustrated in Fig. 60. The shear strength of the soil varied from about 300 to 1000 psf (15-50 kN/m^2).

Stone columns having a diameter of about 3.3 ft. (1 m) were constructed on a square grid. Along the edge of the embankment a stone column spacing of 8 ft. (2.4 m) was used; the spacing was reduced to 5.6 ft. (1.7 m) at interior locations adjacent to the edge. Only sand drains were used on the very interior of the fill. The location and variable spacing used for the stone columns and sand drains are shown in Fig. 61. Approximately 17 percent and 33 percent of the weak soil was replaced by stone in the improved areas for the 8 ft. (2.4 m) and 5.6 ft. (1.7 m) spacing, respectively. The columns, approximately 11 m in length, were backfilled with a granular material composed of 70 percent ballast and 30 percent ungraded sand (Table 6).

A safety factor of 2.0 was used in analyzing the embankment stability. Because of the high strength of the Reinforced Earth, failure circles through it were not considered possible. Also, the strength of the embankment was neglected to consider the possibility of tension cracks.

The project was well instrumented with hydraulic piezometers and Glotzl pressure cells oriented horizontally and vertically and also settlement gages. The total surface settlement was 16 to 20 in. (400-500 mm). Most of the settlement occurred in the upper 22 ft. (6.8 m) of the deposit, being most pronounced in the relatively strong, yet compressible peat layer. Also, the

TABLE 6. GRADATIONS OF STONE USED IN SELECTED STONE COLUMN PROJECTS.

Project	Gradation Percent Passing by Weight				
	3 in. (76 mm)	2-1/2 in. (64 mm)	1 in. (25 mm)	0.5 in. (13 mm)	No. 40 (0.4 mm)
Clark Fork, Idaho	90-100	40-90 (2)	-	0-10 (2)	-
Rouen, France	~100	-	~60	~35	~12
Hampton, Virginia	-	100	65-79 (1)	1-5	-
Santa Barbara, California					
(1) Delivered	85-100	54-97	2-25	0-2	-
(2) Constructed	95-100	86-95	26-40	11-23	3-10

Notes: 1. This size range passes the 1.5 in. sieve.
2. At Clark Fork 40-90 percent passed the 2 in. sieve and 0-10 percent passed the 0.75 in. sieve.
3. Unit Conversions: 1 in. = 25.4 mm.

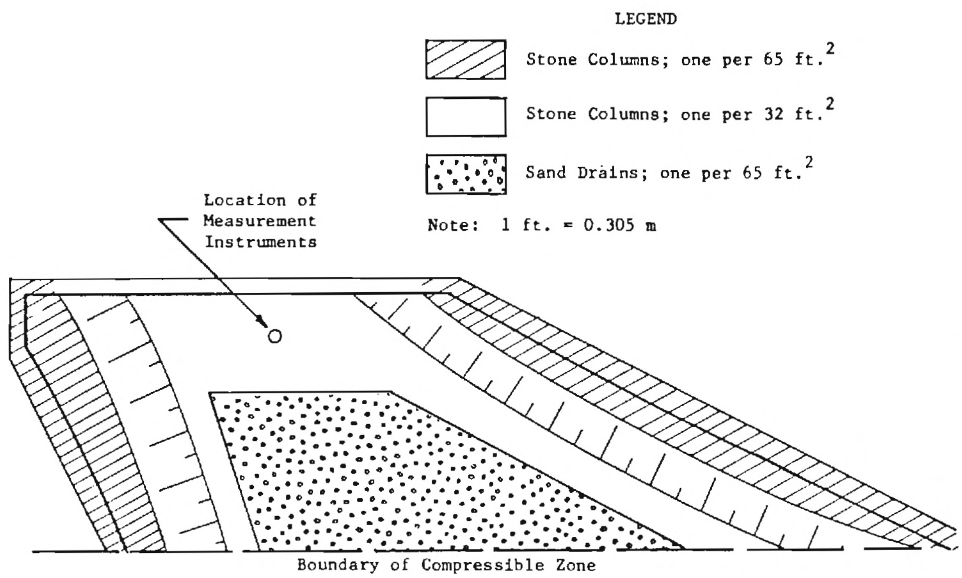

FIGURE 61. PLAN VIEW OF IMPROVEMENT SCHEME AT ROUEN, FRANCE [63].

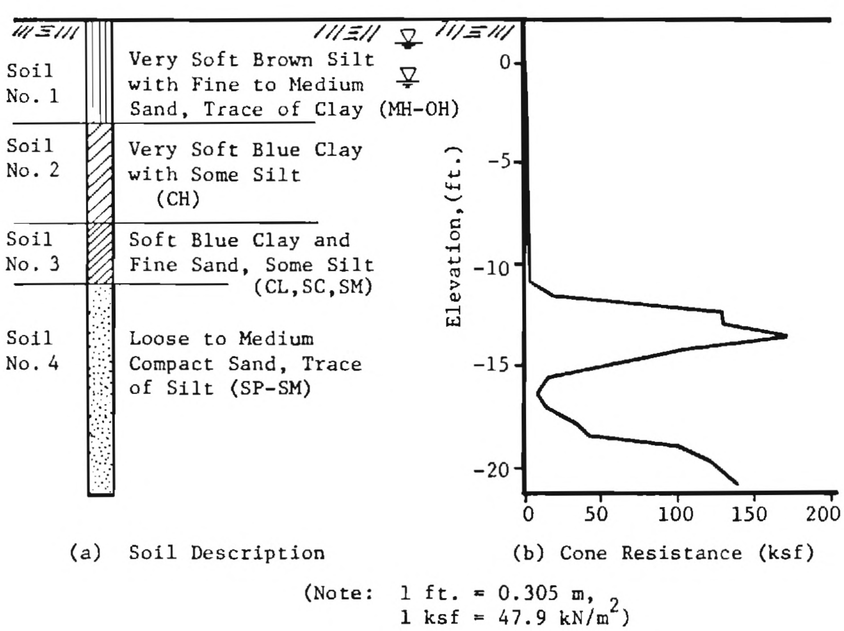

FIGURE 62. SELECTED SUBSURFACE CONDITIONS AT HAMPTON, VIRGINA [27].

stone column and adjacent soil underwent about the same amount of settlement. Approximately 80 percent of the total settlement that occurred during the 7 month monitoring period developed within the first 3 weeks, and 50 percent of the excess pore pressure dissipated within the first month following loading. The stress in the stone columns was found to be approximately 2.5 to 2.7 times the stress in the surrounding compressible soil.

INTERCHANGE EMBANKMENT FILL

Portions of an embankment fill for interchange ramps were supported on stone columns at Hampton, Virginia [27]. Important factors in deciding to reinforce the ground with stone columns included (1) strict environmental constraints, (2) the presence of Newmarket Creek immediately adjacent to the interchange ramp, and (3) achieving acceptable post-construction settlements without delaying the project. Stone columns were selected over (1) excavation and replacement and (2) surcharging due primarily to environmental and economic considerations.

Fill heights in the areas reinforced with stone columns were up to 35 ft. (10.7 m). The subsurface conditions in the vicinity of the stone column supported ramps consisted of 10 to 16 ft. (3-5 m) of erratic, very soft brown silts with sand and very soft to firm, dark gray and blue clays with very thin seams of fine sand and silt as illustrated in Fig. 62. This stratum was underlain by loose to very firm clayey and silty sands, fine to medium sands, and fine sandy clays. The median values of shear strength in the upper 10 to 16 ft. (3-5 m), as determined by field vane tests, were 500 to 600 psf (24-29 kN/m^2), while the median value for the softer zones was about 380 psf (18 kN/m^2). The lowest two values observed at the site were 180 and 200 psf (8.6-9.6 kN/m^2).

To permit working over the very soft marsh, a 3 ft. (0.9 m) sand working platform was first constructed. The stone columns were about 20 ft. (6 m) in length and back-calculated to have about a 3.6 ft. (1.1 m) diameter. The stone columns were carried down into the underlying sands. An equilateral triangular stone column pattern was used; spacing varied from 6 to 8 ft. (1.8-2.4 m). Approximately 18 to 33 percent of the soil was replaced with stone. A 2.5 in. (64 mm) maximum size crushed stone was used having the gradation shown in Table 6. Stone columns were placed beneath the width of the fill along the ramp adjacent to Newmarket Creek within the limits defined by a 60° angle sloping outward from the break in the shoulder. Instrumentation installed in the embankments placed on the stone column improved ground included inclinometers and settlement plates.

To evaluate the use of stone columns at the site before final embankment design, vertical load tests were conducted on a single column (undrained) and also a large group of stone columns (drained). A total of 45 stone columns were constructed in the group load test area; 23 stone columns were beneath and immediately adjacent to the loaded area. The large group was loaded using 401 tons of dead load consisting of precast concrete slabs. The net loading at the original ground surface was 2400 psf (115 kN/m^2). This loading was applied in 54 hours at which time the settlement of the center of the group was 3.1 in. (79 mm); total settlement after 130 days was about 11 in. (300 mm). Sinco piezometers and load cells were used to monitor the load tests. The stress in the stone column at the ground surface was found to vary from about 2.9 to 2.4 times the stress in the adjacent clay.

SEWAGE TREATMENT PLANT-LIQUEFACTION

Stone columns were used to support a large sewage treatment facility at Santa Barbara, California [30,81]. One important design consideration was liquefaction due to seismic activity in the area. Stone columns were selected over (1) excavation and replacement and (2) driven piles largely because of the favorable results of a series of vertical and lateral load tests. The stone columns served the purposes of improving the site to withstand an earthquake having a maximum horizontal acceleration of 0.25 g, and also providing an acceptable vertical load-deformation response when loaded by the sewage treatment facility. Since construction, the plant has safely resisted an earthquake having approximately the design acceleration.

The site was generally underlain by recent estuarine deposits of soft to firm clays and silts, and loose silty sands and clayey sands (Fig. 63). Most of the sandy soils had more than 20 percent passing the No. 200 sieve. The sewage treatment plant was constructed using approximately 280,000 ft. (85,000 m) of stone columns. The design load was 30 tons per column; apparently the assumption was made that no load was carried by the tributary soil. Each column penetrated the recent estaurine deposits into older marine soils; lengths varied from 30 to 49 ft. (9-15 m). A 1 to 3 ft. (0.3-0.9 m) thick distribution blanket of compacted sandy gravel was used to transfer structural loads to the stone columns, and act as a drainage blanket. The thickness of the blanket was varied with the stone column spacing.

The stone column diameters ranged from 2.7 to 4 ft. (0.8-1.2 m) averaging 3.5 ft. (1.07 m). A triangular pattern of stone columns was used. The pattern and spacing varied from a 7 ft. (2.1 m) equilateral triangle to a 4 ft. by 5 ft. (1.2 by 1.5 m) isosceles triangular pattern depending upon the subsurface conditions. The closer spaced grid was used in areas of loose clean sand. About 13 to 32 percent of the soil was replaced by stone.

The stone columns were constructed using both a crushed and uncrushed gravel which was angular to well-rounded. When delivered to the site the stone varied from 0.5 to 3 in. (13-76 mm) in size as shown in Table 6. After construction of a column, however, the gradation was found to be considerably finer with 11 to 23 percent passing the 0.5 in. (13 mm) sieve, and 3 to 10 percent passing the No. 40 sieve (Table 6). The finer gradation resulted from native sand being deposited within the stone column during construction; this phenomenon has been observed at other sites where sand is present.

Twenty eight vertical load tests and direct shear tests on two stone columns were conducted at the site. The results of two vertical load tests are shown in Fig. 64. Load was applied through a circular concrete footing,

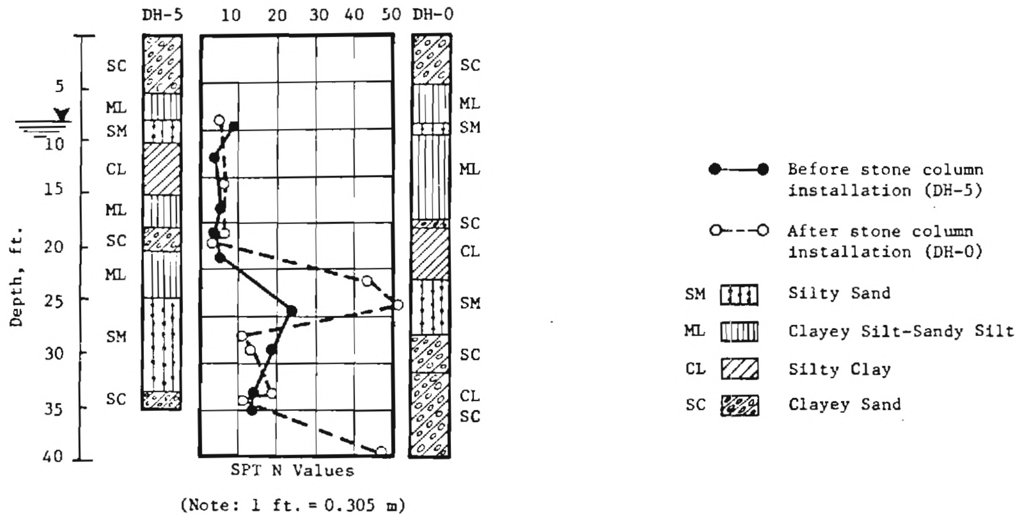

FIGURE 63. SUBSURFACE CONDITIONS AT SANTA BARBARA, INCLUDING EFFECT OF STONE COLUMN IMPROVEMENT ON NATIVE SOIL.

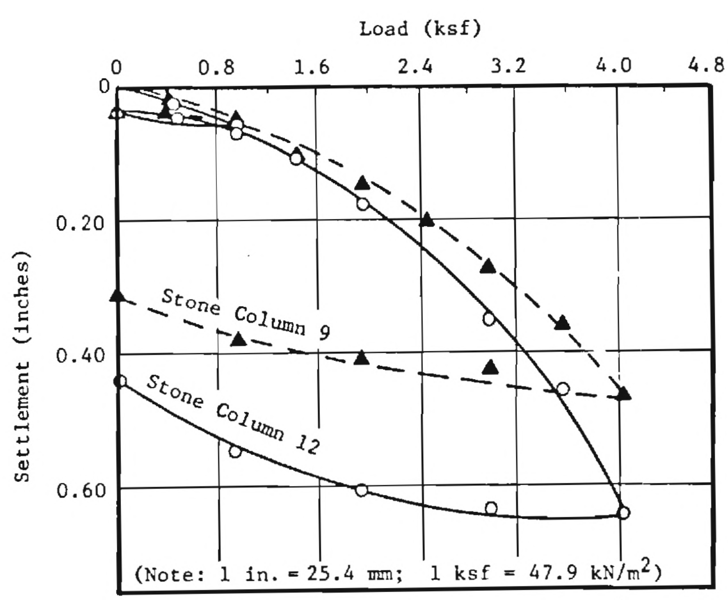

FIGURE 64. VERTICAL LOAD TESTS ON STONE COLUMNS AT SANTA BARBARA.

having a total area equal to that tributary to the stone column. Tests were generally conducted to 1.3 to 1.5 times the design load. Most of the stone columns deflected less than the specified 0.25 in. (6 mm) design criterion under the design load of 30 tons. In areas where a column failed the load test, another load test was performed after constructing additional stone columns.

The ground treatment program was designed to insure ground stability during an earthquake causing a maximum horizontal acceleration of 0.25 g. One assumption used in design was that all of the shear force due to the earthquake would be transmitted through the stone columns, with the surrounding soil contributing no shear resistance. For this condition the safety factor (S.F.) with respect to horizontal shear was evaluated using the expression

$$S.F. = \frac{\tau_s}{a_h \cdot \sigma_s}$$

where: τ_s = shear stress in the stone column that can be mobilized (determined from field direct shear test results at the applied normal stress σ_s acting on the column)

a_h = horizontal earthquake design acceleration coefficient

σ_s = normal stress acting on the stone column

Using the above approach a safety factor was calculated of 3.3. Another design assumption was that the soil and stone column both contributed shear resistance during an earthquake. The corresponding composite shear resistance σ_s was obtained from the results of a direct shear test performed in the field on the combined soil-stone column material present within the tributary area (i.e., unit cell). For this condition the safety factor was found to be 3.4. The final design assumption was that the vertical earthquake acceleration equaled the horizontal design acceleration of 0.25 g. A vertical upward acceleration of 0.25 g effectively reduces the vertical weight and hence stress by 25 percent. Considering a 0.25 g vertical upward acceleration, a safety factor of 2.5 and 2.9 was calculated for the previously discussed conditions of no soil strength and full soil strength, respectively. Apparently, the condition of a vertical downward acceleration of 0.25 g was not considered. The earthquake analyses described above did not consider the loss of strength in the granular materials that might occur due to build-up in the pore pressure during the cyclic earthquake loading. Likewise, possible strength loss in the cohesive soils during cyclic loading was not considered.

Only a few relatively clean sands were encountered at the site that would be highly susceptible to liquefaction. A relatively clean silty sand of this type was found in Test Boring DH-D (Fig. 63) at a depth of approximately 25 ft. (8 m). Standard penetration test results indicated that after stone column construction the relative density of this material was increased to greater than 92 percent. Sands having relative densities of this magnitude are considered not to be susceptible to liquefaction. Stone column spacings were selected using relative density within the unit cell as one criterion.

EMBANKMENT FILL - RIGID STONE COLUMNS

Rigid stone columns were used to support a 25 ft. (7.6 m) high embankment fill for a high speed railway near Munich, Germany [125]. Because of the presence of a thick peat layer conventional stone columns were not feasible. The embankment was constructed immediately adjacent to an existing railway embankment as a result of construction of the Rhine-Main-Danube Canal and highway interchange (Fig. 65).

A typical boring log from the site is shown in Fig. 66. The groundwater table at the site was near the surface. A 1 to 15 ft. (0.3-4.6 m) thick layer of very soft peat having a shear strength of only 100 psf (5 kN/m^2) was encountered at the surface over most of the site. Alternating strata of soft silts and firm clays were found beneath the peat to the boring termination depth of 50 ft. (15 m). A very loose gravel layer 5 to more than 10 ft. (1.5-3 m) in thickness was frequently present at a depth of 6 to 15 ft. (2 to 4.7 m).

Originally, removal and replacement of the peat was planned to increase stability and reduce long-term settlement of the embankment. This alternative involved constructing a temporary sheet pile wall along the edge of the existing adjacent embankment for support during peat removal. The sheet pile wall was to be tied back into the existing embankment. Use of rigid stone columns offered the following advantages over replacement: (1) the sheet pile wall was not required, (2) embankment fill quantities and working area were reduced since the peat was not removed, (3) construction time was decreased, and (4) rigid stone columns offered an economic advantage over replacement.

To stabilize the site, 866 rigid stone columns were constructed using the bottom feed type system previously described in Chapter II. The rigid columns were carried down through the loose gravel strata and terminated in the stiff clay at an average depth of 21 ft. (6.5 m). The design load on each rigid stone column was 45 tons with the measured ultimate load being greater than 130 tons (Fig. 67). The rigid columns varied from 20 to 22 in. (510-560 mm) in diameter. An equilateral triangular pattern of columns was used with the spacing varying from 5.2 to 7.2 ft. (1.6-2.2 m). Each rigid column had a total tributary area of 30 to 42 ft^2 (2.8-3.9 m^2) depending upon the embankment height. The corresponding area replacement ratios a_s varied from 0.06 to 0.08, which is much less than usually used for conventional stone columns. Reported settlement of the embankment was less than 0.25 in. (6 mm).

The rigid stone columns were constructed using a ready mix concrete which was pumped to the bottom of the hole through the small feeder pipe attached to the outside of the main vibrator tube. The feeder pipe was approximately 4.75 in. (120 mm) in diameter. The concrete had a maximum aggregate size of 1.25 in. (32 mm), and an unconfined compressive strength of 5,000 psi (34,000 kN/m^2). After pushing the probe to the final elevation

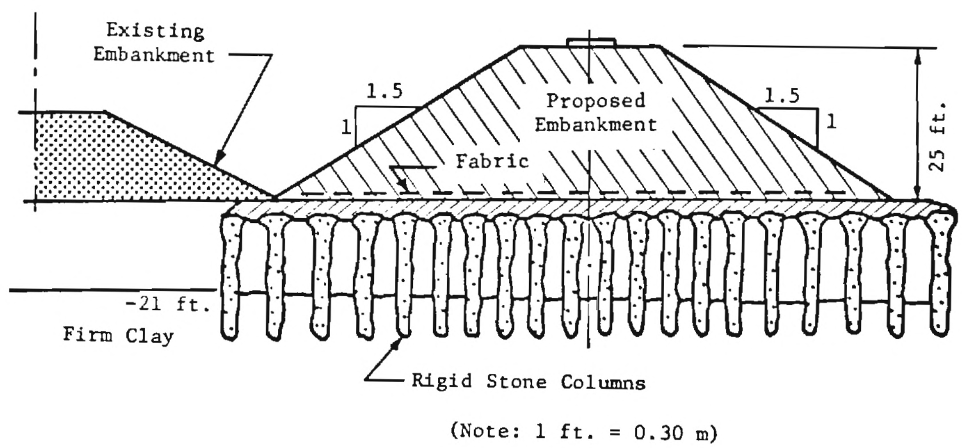

FIGURE 65. EMBANKMENT SECTION AT MUNICH, GERMANY - RIGID STONE COLUMNS [125].

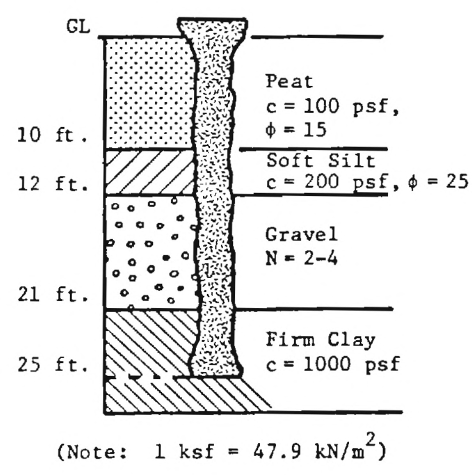

FIGURE 66. TYPICAL SOIL PROFILE AT MUNICH, GERMANY - RIGID STONE COLUMNS [125].

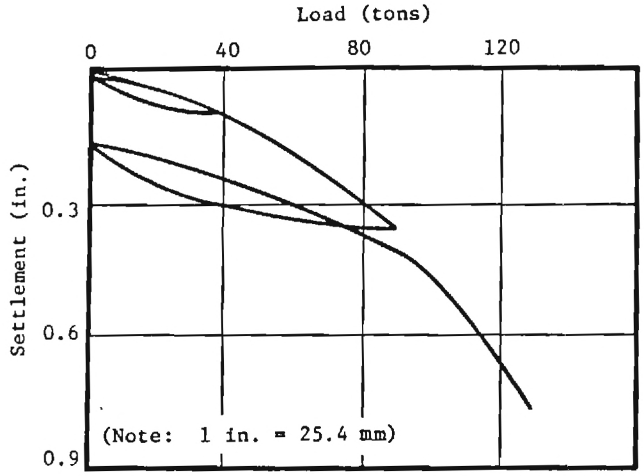

FIGURE 67. VERTICAL LOAD-SETTLEMENT RESPONSE OF RIGID STONE COLUMN AT MUNICH, GERMANY [125].

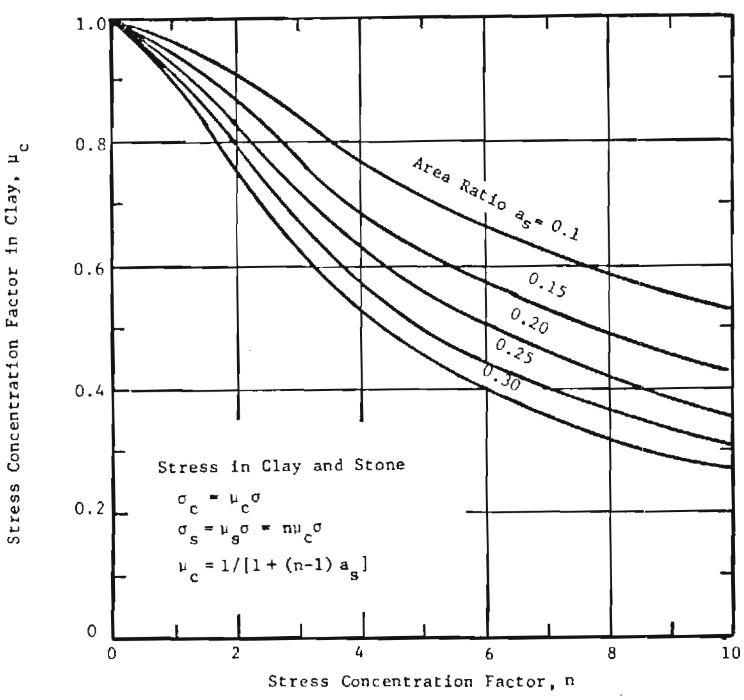

FIGURE 68. VARIATION OF STRESS IN CLAY WITH STRESS CONCENTRATION FACTOR AND AREA RATIO.

with the vibrator running, the tubes were lifted about 1 ft. (0.3 m). Enough concrete was then pumped into the bottom to fill this space, and the concrete was repenetrated by the vibrator. The tube was slowly and continuously withdrawn (with the vibrator running) as concrete was pumped into the hole left by the tube. Running the vibrator as the tube was withdrawn densified the concrete and pushed it into the surrounding soil. A rigid column constructed in this way is quite similar to a conventional cast-in-place concrete or auger cast pile. Conventional piles, however, are not subjected to the high level of vibration that a rigid stone column undergoes.

A 1 to 2 ft. (0.3-0.6 m) thick granular blanket was placed over the rigid columns. A fabric layer having a tensile strength of 1 to 2 tons/ft. (3-6 tons/m) was laid at the interface between the granular blanket and the embankment to resist horizontal embankment forces. Use of a granular blanket and fabric over rigid stone columns is a common practice in Germany.

SUMMARY

Five selected case histories were briefly described of applications of stone columns. A careful study of such case histories provides valuable insight into the present state-of-the-art of stone column practices including the utilization of distribution blankets, load tests, field monitoring, and performance and design features such as stone column diameter, spacing, area replacement ratio, and design load. The stone gradations used in most of these projects are summarized in Table 6. Of interest is the finding that the gradation of a stone column may be significantly finer after construction at sites where native sand is present in the strata penetrated by the stone columns.

CHAPTER VII

SYNTHESIS OF RESULTS–DESIGN OF STONE COLUMNS

INTRODUCTION

The purpose of this chapter is to synthesize the results of this study from the design viewpoint. In practice the design of stone columns for ultimate capacity is to a large extent empirical; for settlement the design is, to varying degrees, less empirical. Specific state-of-the-art design recommendations are given for bearing capacity, settlement and stability analyses. Bridge bent and abutment design using stone column improved ground is also discussed. These design recommendations give a rational basis upon which to evaluate stone columns. Theoretical results, of course, should always be supplemented by past experience and sound engineering judgement. Certainly much theoretical research and particularly field verification remains to be done.

STRESS CONCENTRATION

Stress concentration due to an overburden load above the stone column causes an increase in shear strength in the column, and reduction in settlement in the surrounding soil. Stress concentration occurs because the stone column is considerably stiffer than the surrounding soil. Since the deflection in the two materials is approximately the same, from equilibrium considerations the stress in the stiffer stone column must be greater than the stress in the surrounding soil. The assumption of equal deflection is frequently referred to as an equal strain assumption, for example, in time rate of consolidation theory. Both field measurements made by Vautrain [63] and the finite element analyses conducted as a part of this study indicate the equal strain assumption is realistic.

The stress concentration factor n is the ratio of the average stress in the stone column σ_s to the stress σ_c in the soil within the area tributary to the column (Fig. 14c). The stone column and tributary area comprise the unit cell. Equations (8a) and (8b) are used to calculate the average stress in the tributary soil and stone column, respectively. *Stress concentration is a very important concept which accounts for much of the beneficial effect of improving marginal ground with stone columns.* For comparative purposes the influence of the stress concentration factor on the stress in the soil and stone can be easily determined using Fig. 68.

Field Measurement

The stress concentration factor n is dependent upon a number of variables including the relative stiffness between the two materials, length of the stone column, area ratio and the characteristics of the granular blanket placed over the stone column. Values of stress concentration measured in field and laboratory studies are summarized in Table 7. Measured values of stress concentration have generally been between 2.5 and 5.0. The stress concentration factor measured in 4 of the 5 studies was either approximately constant or increased with time as consolidation occurred. Theory indicates the concentration factor should increase with time [57]. Since secondary settlement in reinforced cohesive soils is greater than in the stone column, the long-term stress concentration in the stone column should be no less than at the end of primary settlement. Field measurements for sand compaction piles at four sites in Japan [24] indicated stress concentration probably decreased with depth, but remained greater than 3.0 at the sites studied.

ULTIMATE BEARING CAPACITY OF STONE COLUMNS

In determining the ultimate bearing capacity of a stone column or a stone column group, the possible modes of failure should be considered as illustrated in Figs. 5, 9, and 12. Particular caution should be given to avoiding local bulging failures due to very weak, potentially organic, layers of limited thickness (Fig. 12). Bulging would have a great effect upon settlement; bulging would also be of concern with respect to stability. Use of a bulging analysis for a single column to predict group behavior gives admittedly an approximate solution which may be conservative. A discussion of the failure modes and theory for calculating the ultimate bearing capacity of stone columns was given in Chapter III.

Design

The rational prediction of the bearing capacity of stone column groups loaded by either a rigid foundation or a flexible load due for example to an embankment is still in the developmental stage. As a result, past experience and good engineering judgement should be used in addition to theory when selecting a design stone column load.

Single Column Analysis. Frequently the ultimate capacity of a stone column group is predicted by estimating the capacity of a single column and multiplying that capacity by the number of columns in the group. Small scale model studies using a rigid footing indicate this approach is probably slightly conservative for soft cohesive soils. The bearing capacity of an isolated stone column or a stone column located within a group can be expressed in terms of an ultimate stress applied over the stone column:

$$\tilde{q}_{ult} = c\tilde{N}_c \tag{50}$$

TABLE 7. OBSERVED STRESS CONCENTRATION FACTORS IN STONE COLUMNS[1].

Type Test	Design	Location	Stress Concentration n	Time Variation of n	Stone Col. Length (ft)	Subsurface Conditions
Embankment	Square Grid S = 5.7 ft. D = 3 ft.[2] a_s = 0.25	Rouen, France- Vautrain [63]	2.8 (avg.)	Approx. Constant	22-26	Soft clay: c = 400-600 psf
Load Test; 45 stone columns (36'x50')	Triangular Grid; S = 5.8 ft. D = 4.0 ft.[2] a_s = 0.43	Hampton, Virginia- Goughnour, et al [27]	3.0 (initial) 2.6 (final)	Decreasing	20.5	Very soft and soft silt and clay with sand; c = 200-800 psf
Test Fill 14 stone columns[3]	Triangular Grid; S = 7 ft. D = 3.75 ft. a_s = 0.26	Jourdan Road Terminal, New Orleans, La. [71]	2.6-2.4 (init.) 4.0-4.5 (final)	Increasing	65	Very soft clay with organics, silt and sand lenses; loose clayey sand; soft sandy clay.
Embank- ments	a_s = 0.1- 0.3	Japanese Studies - Sand compac- tion piles[5] Aboshi, et al [24]	2.5-8.5 4.9 (average)	Increases	Variable	Very soft and soft sediments
Model Test	a_s = 0.07-.4 D = 1.14 in.	GaTech Model Tests; Unit Cell; Sand Column	1.5 - 5.0	Constant to Slightly Increasing	Variable	Soft Clay; n appears to increase with a_s

Notes: 1. Vertical stress measured just below load except where indicated otherwise.
2. The diameter and area ratio a_s are based on a stone density of 105 pcf (16.5 kN/m^3).
3. Eight additional stone columns were installed in the wing walls.
4. Measured at the end of the 15 week consolidation period.
5. Stress concentration measured at 12 sites; at 4 sites stress concentrations were measured at depths of 10 to 49 ft.
6. Unit Conversions: 1 ft. = 0.305 m; 1 in. = 25.4 mm; 1 psf = 47.9 N/m^2.

where: $\tilde{q}_{ult}$ = ultimate stress which the stone column can carry

c = undrained shear strength of the surrounding, cohesive soil

$\tilde{N}$ = bearing capacity factor for the stone column ($18 \lesssim \tilde{N}_c \lesssim 22$)

The ultimate bearing capacity of the tributary soil can be taken as 5c with an upper limit of $\mu_c\sigma$. In evaluating $\tilde{N}_c$ from field test results stress concentration should be considered using equation (8).

Cavity expansion theory shows that the ultimate capacity and hence $\tilde{N}_c$ is dependent upon the compressibility of the soil surrounding the stone column. Hence soils having organics, for example, would be expected to have a smaller value of $\tilde{N}_c$ compared to better soils. For soils having a reasonably high initial stiffness an $\tilde{N}_c$ of 22 is recommended; for soils with low stiffness, an $\tilde{N}_c$ of 18 is recommended. Low stiffness soils would include peats, organic cohesive soils and very soft clays with plasticity indices greater than 30. High stiffness soils would include inorganic soft to stiff clays and silts. The recommended values of $\tilde{N}_c$ are based on a back-analysis of field test results (Chapter III, Fig. 49). In this analysis the strength of both the soil and stone column was included.

Mitchell [67] recommends using an $\tilde{N}_c$ of 25 for vibroreplacement stone columns. Datye, et al. [73] recommend using 25 to 30 for vibroreplacement columns, 45 to 50 for cased, rammed stone columns and 40 for uncased, rammed stone columns. Wallays [51] has also found that rammed stone columns apparently have higher ultimate capacities than vibroreplacement stone columns. Of course, the equipment, experience, and construction technique used have a significant influence on the performance of all type stone columns. The above values of $\tilde{N}_c$ can be used, without including the strength of the surrounding soil, to estimate the lower limit of the load which the improved ground can carry; such an analysis is most applicable for problems such as foundations where settlement is of great concern.

Group Bearing Capacity Theory. The group bearing capacity theory presented in Chapter III, equations (16)-(19), offers a valuable tool for analyzing the ultimate capacity of small stone column groups constructed in cohesive soils. The group is assumed to be loaded by a rigid foundation. In the development of the group bearing capacity theory for rigid foundation loading, a general shear failure consisting of a straight failure surface was assumed to occur in the composite stone-cohesive soil mass beneath the foundation. The possibility of a local bulging failure of individual stone columns was not considered in the analysis. Therefore this theory is applicable for firm and stronger cohesive soils having undrained shear strengths greater than about 600 to 800 psf (30-40 kN/m^2). The group theory is useful for determining, at least approximately, the relative effects on ultimate capacity of design variables such as stone column diameter and spacing, increase in shear strength due to consolidation and angle of internal friction.

In softer cohesive soils both model and full-scale tests indicate the full shear strength of the stone column and surrounding soil may not always be mobilized. Field direct shear tests conducted at Santa Barbara, California and Jourdan Road Terminal (to be described subsequently) indicate a significant reduction may occur in friction angle accompanied by an increase in the apparent cohesion of the combined soil-stone column mass. Therefore, for the present time the ultimate capacity of foundations constructed on soft and very soft cohesive soils should be predicted using equation (50). The occurrence of soft and very soft cohesive layers at depth can be approximately considered using the approach illustrated in Bearing Capacity Example 1, Appendix C.

Cavity Expansion Theory. Vesic cavity expansion theory [61], equations (12)-(14), is recommended primarily for use with the group bearing capacity theory to calculate the confining pressure for a square group as illustrated by Bearing Capacity Example 2, Appendix C. The theory could, however, be used for other applications. For use in Vesic cavity expansion theory, a modulus E of 11c is recommended for soft to stiff, non-organic soils. For organic or very soft soils with a plasticity index greater than 30, an E of 5c is recommended. An angle of internal friction of 42 to 45^{o} should be used in the analysis for a good quality crushed stone and 38 to 42^{o} for a gravel.

Design Recommendations

For routine design the ultimate capacity of a group of stone columns should be estimated using equation (50) following the recommendations given previously. Where bearing capacity is critical such as for embankments or heavy tanks, a circular arc stability analysis should be used to analyze the overall stability. A circular arc analysis would give, because of the presence of end effects, conservative results for square foundations and rectangular foundations having length to width ratios less than 5 to 10. For projects where bearing capacity is critical, the increase in shear strength due to consolidation can also be considered using the method given in Chapter III, or the more sophisticated approach of Ladd and Foott [102].

Locally soft layers often exist at some depth beneath the surface. For such conditions, the possibility of a local bulging failure of individual piles (Fig. 12), should be analyzed using the method illustrated in Appendix C, Bearing Capacity Example 1.

For the design of embankments, tanks, and similar structures, a mininum safety factor of 1.5 to 2.0 is recommended with respect to a bearing capacity failure. Where important, settlement should also be considered. In many instances settlement considerations will limit the load that can be applied to the stone column improved ground.

SETTLEMENT PREDICTION

Primary Consolidation Settlement

Methods for estimating settlement of stone columns were presented in Chapter III. For very soft to firm cohesive soils reinforced with stone columns, a best estimate of settlement should be made by bounding the answer. For the upper bound, the equilibrium method (equation 20) is recommended, while the nonlinear finite element design curves, (Figs. 29 to 37) should be used for the lower bound. In general, the best settlement estimate should then be taken as being the average of the two estimates. For comparison, a settlement estimate using the incremental method could also be performed. A comparison of predicted and measured settlements is given in Table 8.

For settlement calculations using the equilibrium method, a stress concentration factor n of 4.0 to 5.0 is recommended based on comparisons of calculated settlement with observed settlements at Hampton and Jourdan Road Terminal. Also, at one site in Japan, Aboshi, et al. [24] found for sand compaction piles n = 4 gave the best agreement for a site having a measured settlement of 6.6 ft. (2 m).

With the finite element method, the solutions for the soft boundaries should be used for soils having shear strengths less than 600 psf (40 kN/m^2). For soils having shear strengths between 600 and 1000 psf (40-70 kN/m^2), interpolation should be used between the soft and rigid boundary curves. For cohesionless soils and very stiff to hard clays reinforced with stone columns the elastic solutions (Figs. 24-26) can be used provided the modular ratio of the stone column to the soil is less than 10.

Drained Modulus of Elasticity of Cohesive Soil. For use in the finite element approach, the drained modulus of elasticity E_c of the cohesive soil can be obtained from either drained triaxial tests or from one-dimensional consolidation tests. Theoretically, drained triaxial tests should give the best estimate of E_c since this test allows three-dimensional deformation. Also, the drained test gives the modulus E_c directly in contrast to the consolidation test which requires back-calculation of E_c. Many laboratories are not, however, equipped to perform a long-term drained test. A consolidated undrained test is not in general recommended for evaluating E_c since the effective confining pressure continuously varies throughout the test. Laboratory testing which follows the anticipated stress path of the soil upon loading could be performed, but in general, the elaborate nature of the testing and monitoring would not be justified.

The recommendation is therefore given that the drained modulus of elasticity be back-calculated from the results of one-dimensional consolidation tests using equation (47) of Chapter III. This approach is both practical and has the important advantage that a number of consolidation tests can be performed to give a representative variation of E_c within a given stratum. The modulus of elasticity calculated is dependent upon the average stress used in equation (47). Therefore, care should be exercised to use the average of the initial effective stress before construction and the effective stress in the cohesive soil ultimately developed after primary consolidation. The change in stress in the cohesive soil due to construction can be estimated using equation (8a). Typical values of drained Poisson's ratio for use in calculating E_c are given in Table 9. The equilibrium and the finite element methods and also the incremental method all require performing consolidation tests. The number of tests required varies with the geologic site conditions and the importance of the settlement estimate. A minumum of 8 to 10 tests is recommended as a very general guide

TABLE 8. COMPARISON OF CALCULATED AND MEASURED SETTLEMENTS AT HAMPTON, VIRGINIA AND JOURDAN ROAD TERMINAL, NEW ORLEANS.

Location	Test Type	Stone Column Design				Settlement (inches)		Measured Primary Consolidation Settlement
		Loading σ (ksf)	Dia. (ft)	L/D	a_s	Equilibrium Method[2] (in.)	Finite Element (in.)	
Hampton, Virginia (c = 460 psf)[4]	Test Group (45 col.)	2.4	3.6	5	0.34	18 (n = 5)	9 (ν_o = 0.43)	12
Hampton, Virginia (c = 350 psf)[4]	Embankment Fill (44' wide)	0.9	3.6	~4	0.24	17 (n = 5)	12 (ν_o = 0.43)	~15
Jourdan Road (c = 200 to 500 psf)[5]	Test Fill (14 col.)	1.2	3.75	~16[7]	0.24	25 (n = 5)	14 (ν_o = 0.43)	~14 - 16[6]

Notes: 1. A ν_c = 0.43 was used to calculate E_c for the nonlinear finite element analysis; soft boundary conditions were also used on the design curves.

2. Using an n = 4 increases the settlement by 1 to 2 in. for these examples.

3. Embankment 44 ft. wide at top; 7.35 ft. high; 2:1 side slopes; stone columns over 52.8 ft. base width.

4. Shear strength from field vane shear tests.

5. Shear strength from unconfined compression tests; shear strength increases with depth.

6. A significant amount of secondary compression settlement occurred making the estimation of primary consolidation settlement difficult.

7. The settlement estimate was based on a L/D = 12.2 to consider the better soils encountered depth.

TABLE 9. TYPICAL POISSON'S RATIO VALUES OF CLAY FOR DRAINED LOADING [119].

Soil Consistency	Poisson's Ratio[1]
Very Soft to Soft[2,3]	0.35 - 0.45
Firm to Stiff[2]	0.30 - 0.35
Stiff Overconsolidated Clays	0.1 - 0.30

Notes:
1. For undrained loading use 0.45.
2. For normally consolidated clays.
3. For very soft to soft clays a value of 0.40-0.45 is recommended for calculating E_c for nonlinear finite element settlement analyses of stone column improved ground; for firm to stiff use at least $\nu_c = 0.35$.

within the worst stratum; fewer tests can be performed within the better layers.

Modulus of Elasticity of Stone Column. Both the incremental and elastic methods require the modulus of elasticity E_s of the stone column. By back-calculation using measured field settlements, Vautrain [63] determined E_s actually developed was about 4,400 psi (30,000 kN/m^2) for the vibroreplacement stone columns at Rouen. Balaam [57] estimated E_s to be 7,200 psi (50,000 kN/m^2) from the linear portion of the undrained load settlement curve obtained at Canvey Island. Englehart and Kirsh [57] recommend using a value of 8,400 psi (58,000 kN/m^2). For rammed stone columns Datye, et al. [73] found by back-calculating from measured settlements that E_s was 7,000 psi (48,000 kN/m^2).

The modulus of elasticity of the stone column varies with the state of stress developed within the column both during and after construction. Because of greater confinement, long stone columns should therefore have a greater average modulus of elasticity than short columns. The modulus of elasticity E_s of the stone column can be calculated using

$$E_s = (\sigma_1 - \sigma_3)/\varepsilon_a \tag{51}$$

where: $\sigma_1 - \sigma_3$ = deviator stress under the applied loading
σ_1 $\cong$ vertical stress in stone column
σ_3 $\cong$ lateral stress in stone column

Both the initial at-rest stress in the stone column and the change in stress due to loading should be used in calculating σ_1 and σ_3. The axial strain ε_a can be obtained directly from the stress-strain curves for the stone obtained from triaxial shear tests.

In the absence of field load test or triaxial test results, the modulus of the stone can be estimated using the hyperbolic expression developed following the approach of Duncan and Chang [117]

$$E_s = K\,\sigma_\theta^n \left[1 - \frac{(\sigma_1 - \sigma_3)R_f}{\left(\dfrac{2(c \cdot \cos\phi_s + \sigma_3 \sin\phi_s)}{1 - \sin\phi_s}\right)}\right] \tag{52}$$

where: E_s = stress dependent secant modulus of the stone
K,n = constants defining the initial modulus of the stone (under low deviator stress)
c = cohesion of the stone (normally taken as zero)
ϕ_s = angle of internal friction of the stone
R_f = failure ratio
σ_θ = $\sigma_1 + \sigma_2 + \sigma_3$

In the absence of specific test data, the following constants can be used for soft clays: K = 88.6, n = 1.14, R_f = 0.86, c = 0, and typically $\phi_s \cong 42°$ to $45°$ where σ_θ and E_s are in psi; these constants give a response similar to that used to develop the settlement curves (Figs. 29-37). Use of these constants in equation (52) typically gives values of E_s in the range of 1000 to 3000 psi (7000-21,000 kN/m^3), which is less than the modulus usually assumed. Since in a soft clay the stone column is in a state of failure, a high deviator stress and low confining pressure exists in the stone. Therefore, the existence of a low modulus for the stone is possible.

A K_o value of 0.5 to 1.0 is recommended for calculating initial lateral stress in the stone due to weight. Finite element analyses indicate the lateral stress due to the applied loading can be calculated using equation (9) for soft to very soft soils. Of course, stress concentration should be considered.

The finite element study indicates values of E_s/E_c for soft cohesive soils up to about 100 for vibro-replacement stone columns. This range in modular ratio extends above the upper limit of 40 suggested by Balaam, et al. [78]; Datye, et al. [73] indicate the lower limit of the ratio is 100.

Time Rate of Settlement

Stone columns act similarly to sand drains in decreasing the distance which water has to flow in the radial direction for primary consolidation to occur. As a result installation of stone columns can, in the absence of natural drainage layers within cohesive soils, significantly decrease the time required for primary consolidation. Under these conditions, the presence of stone columns will greatly accelerate the gain in shear strength of the cohesive soil as primary consolidation occurs. The presence of permeable sand seams, partings, lenses, or layers will, however, decrease or even eliminate the beneficial effect of the stone columns in accelerating primary consolidation. Past experience has shown that the actual rate of consolidation occurring in the field is usually faster than predicted [87].

The time rate of primary consolidation settlement should be estimated using the sand drain consolidation theory presented in Chapter III and summarized in Figs. 42 and 43. The horizontal permeability of many strata in which stone columns are constructed is likely to be 3 to 5 times or more the vertical permeability. Further, constructing the stone column results in a reduction in horizontal permeability near the stone column due to what is usually referred to as "smear effects" which includes smear of the surrounding soil during construction, remolding, and intrusion of soil into the voids of the stone column near the periphery. In predicting time rate of settlement, the effects of smear can be correctly handled mathematically using a reduced drain diameter [85]. Use of a reduced drain diameter rather than a smear factor permits a physical feel for the effects of smear. The correct reduced drain diameter is chosen using Fig. 44 after the smear factor is calculated.

Relatively little is known about the effect of smear on the time rate of consolidation for sand drains [87]; even less is known about smear effects for stone column applications. To approximately consider the effect of smear, the radius r_w of sand drains is sometimes halved [88, p. 175]. Goughnour and Bayuk [27] have performed a comprehensive analysis of the results of the Hampton, Virginia load tests on stone columns. Assuming the horizontal permeability to be three times the vertical permeability, a smear factor of 2.5 was found by Goughnour and Bayuk to give a good approximation of the measured time rate of settlement. A smear factor of 2.5 is equivalent to dividing the actual stone column radius by about 18. Bidhe and Datye have used an equivalent smear factor of 0.1.

An analysis of the Jourdan Road load test results suggests that the smear factor was probably less than 0.6, which corresponds to using one-half the radius of the stone columns. At Jourdan Road the presence, however, of roots, humus, wood, sand lenses and layers and shells makes a reliable time rate of consolidation analysis impossible. As a result of the favorable drainage conditions at the site, primary consolidation occurred very quickly.

In the absence of other data on the effects of smear, a reduction in diameter of from 1/2 to 1/15 of the actual diameter is tentatively recommended based, admittedly, on meager data. Certainly more research is needed to establish reliable procedures for determining the appropriate reduction in stone column diameter to account for smear.

For routine projects laboratory permeability tests should be performed to evaluate the horizontal permeability of the compressible stratum (refer to Chapter IV). A careful examination of the undisturbed samples, grain size tests, and site geology can also be used as a guide in estimating the ratio between horizontal and vertical permeability. In the absence of better data, a coefficient of horizontal permeability of 3 to 5 times the vertical coefficient of permeability can be assumed in the analysis. The coefficient of consolidation can be calculated once the permeability is established using equation (49).

Some non-routine stone column projects will be encountered where reliable estimates of time rate of settlement are necessary for the success of the project or for a reliable comparison of design alternatives. For such projects, the horizontal permeability should be evaluated using field pumping tests. Piezometer or well point permeability tests are alternatives which should give horizontal permeabilities equal to or less than those obtained from pumping tests. If vibro-replacement is to be used, the drains and well points to be used for permeability tests should be installed by jetting; driving which causes smear should not be permitted.

Secondary Settlement

The theory for predicting secondary settlement was given in Chapter III, and is summarized by equation (30). Secondary settlement calculated using the theory should be considered as only a rough estimate [88].

Secondary settlement equal to or even greater than primary consolidation settlement can occur in highly organic soils and some soft clays; important secondary settlement can also occur in highly micaceous soils [74,88]. Highly organic soils and soft clays are likely candidates for reinforcement with stone columns to support embankment loads. Secondary compression settlement will therefore be an important consideration in many stone column projects. Because of the relatively short time usually required for primary consolidation to take place in stone column reinforced soils, secondary settlement is even more important than if drains are not used.

Neither stone columns nor sand drains accelerate the time for secondary settlement. For example, in one instance sand drains were used to accelerate settlement beneath a highway embankment [87]. The subsurface conditions consisted of 5 ft. (1.5 m) of fibrous organics and organic silt overlying 20 to 25 ft. (6-8 m) of soft, dark clayey silt. Primary consolidation was complete by the end of construction. Nevertheless, by the end of 4 years the pavement had been resurfaced twice, with secondary settlements reaching 1 ft. (0.3 m).

Rutledge and Johnson [87] indicate that based on field observation, theory, and laboratory tests, the secondary compression can be reduced to tolerable levels by surcharge loading. The amount of secondary compression that occurs is directly related to the level of the surcharge. To be effective the surcharge must apply an effective stress greater than will be ultimately reached under the service loading. Areas of greatest differential settlement of course are of most concern. For sites where secondary settlement is important, consideration should be given to surcharge loading at least at transitions from areas of small to great settlements such as bridge abutments and transitions to firm strata.

STABILITY

Design

An important use of stone columns is to improve marginal sites to permit construction of embankments; stone columns can also be used to stabilize existing slopes. These applications both involve improving the overall stability of the loaded soil mass and require stability analyses. For homogeneous or erratic soil conditions where a circular arc type failure is likely to occur, the Simplified Bishop method of stability analysis should be performed. For soil conditions where a linear failure will occur such as at sites where thin, continuous weak layers or varved clays are encountered, the Morgenstern-Price Method is recommended. A good review of slope stability methods has been given elsewhere [89,90]. Standard computer programs such as LEASE [91,122] and STABL [123] are available for solving stability problems using both the Simplified Bishop and Morgenstern-Price Methods.

The stone columns should be laid out to minimize the number of columns required to give the necessary overall safety factor for any possible failure surface. Stone column spacing (area replacement ratio, a_s) can, to some degree, be varied to achieve a balanced design with respect to embankment safety. For example, an embankment having a maximum height of 28 ft. (8.5 m) located at Rouen, France had a varying stone column spacing, and also utilized sand drains on the interior as shown in Fig. 61. Wick drains could have been used instead of sand drains. The normal stress acting on the stone column is a maximum beneath the center of the embankment. Other factors being equal, stone columns located beneath the embankment can develop a greater resisting moment and hence work more efficiently than if placed outside the toe. The stone columns should therefore be concentrated under the embankment as much as practical to achieve the highest efficiency.

Typically area replacement ratios of 0.15 to 0.35 are used to improve stability at marginal sites; this means 15 to 35 percent of the weak material is replaced by stone columns. For low levels of replacement and modest fill heights, variations in the values of n and ϕ_s used in a stability analysis may have a relatively small effect on the overall stability of the mass. For example, at the Jourdan Road Terminal [71] stability test fill, the area replacement ratio used was about 0.1 and shear strength of the soil 300 to 400 psf (14-19 kN/m^2). Stability analyses indicated increasing the angle of internal friction of the stone from 38° to 45°, and increasing n from 2 to 3.5 both caused only a 5 percent increase in safety factor. One reason for the low effect of the stone columns was the relatively small embankment height which caused the development of low shearing resistance in the column. Had the shear strength of the soil been greater, the effect would have been even less. An increase in shear strength from 300 to 400 psf (14-19 kN/m^2) caused a 21 percent increase in safety factor. This finding indicates the important improvement that can be obtained using stage construction either with or without stone columns. Certainly stage construction or stability counter berms should be carefully considered as design alternatives, particularly for soft cohesive soils and moderate fill heights. A stability example using stone columns is given in Appendix E.

Composite Action/Direct Shear Tests

Field, laboratory, and theoretical results indicate that the full shear strength of the stone column and surrounding soil may not always be mobilized within the unit cell when the shear strength of the soil is less than about 600 to 800 psf (30-40 kN/m^2). Analyses and design curves for local bearing failure of a single column are given in Appendix B.

Direct shear tests were performed in the field at Santa Barbara [30] on a 3.5 ft. (1.07 m) diameter stone column acting together with its tributary soil. The stone column and soil were enclosed by a single steel ring as shown in Fig. 69. For normal stresses greater than about 1500 psf (72 kN/m^2), the measured shear strength of the combined soil mass was less than that of the stone column alone (Fig. 70). Composite action of the stone column-surrounding soil together with local bearing appear to account for this reduction in strength.

In the past direct shear tests conducted in the field have been performed using only a single steel ring to form the upper part of the shear box. Below the failure surface, the stone column has reacted directly against the surrounding soil as illustrated in Fig. 69. Generally, the shear load has been applied using a hydraulic jack reacting against an adjacent stone column. At Jourdan Road Terminal when this type direct

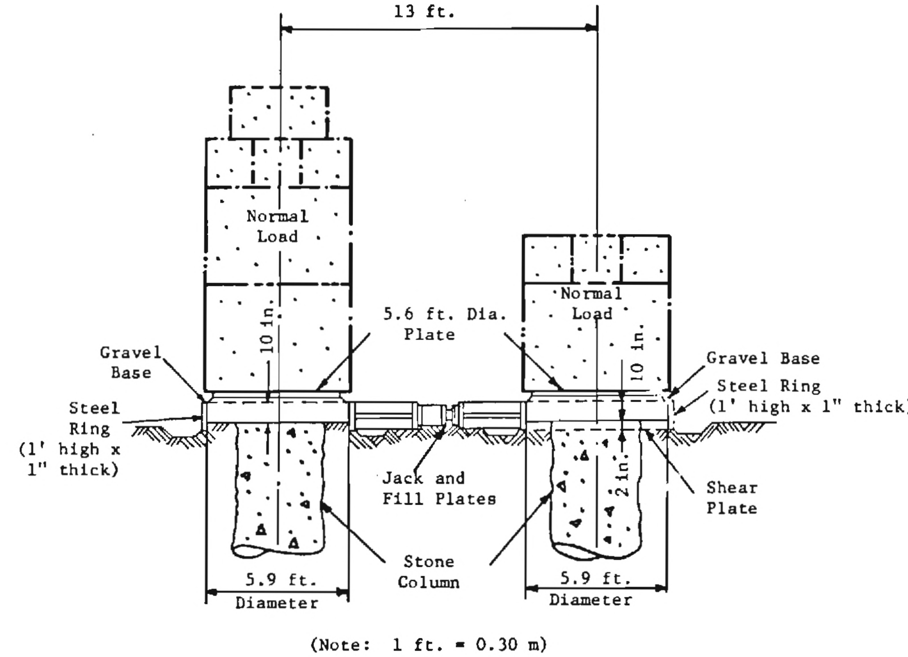

FIGURE 69. LATERAL LOAD TEST SET-UP USED AT SANTA BARBARA SEWAGE TREATMENT FACILITY [30].

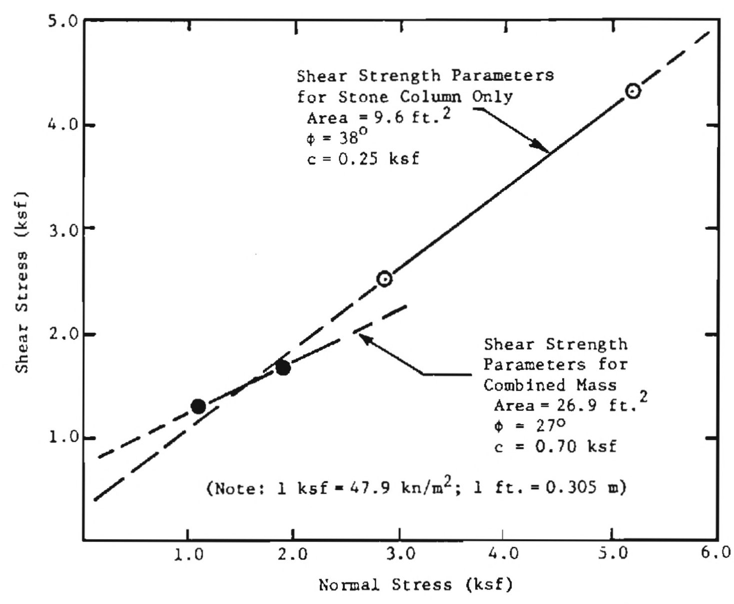

FIGURE 70. COMPARISON OF SHEAR STRENGTH OF STONE COLUMN AND COMBINED MASS - SANTA BARBARA [30].

shear test was conducted, a local bearing failure was observed to occur behind the stone column, resulting in a quite low angle of internal friction $\phi = 21°$ and cohesion $c = 260$ psf (12.5 kN/m^2). These strength parameters reflected the combined strength of the stone column and the surrounding soil and not the strength of just the stone column.

To prevent a local bearing failure and bending, another direct shear test was performed at Jourdan Road Terminal using an upper and lower steel ring to form a direct shear box (Fig. 71). This type loading arrangement prevented a local shear failure and showed the in-situ shear strength of just the stone to be 54°. The strength envelopes for the single and double ring shear tests are compared in Fig. 72.

At Steel Bayou [111] direct shear tests were also performed in the field on 3 ft. (0.9 m) diameter stone columns constructed using a gravel. A composite shear strength of $\phi = 33°$ and $c = 425$ psf (20 kN/m^2) was obtained from the single ring shear test (Table 10). Undoubtedly bearing of the stone column against the surrounding soil significantly influenced these test results. Direct shear tests later conducted in a 2 ft. by 2 ft. (0.6 by 0.6 m) direct shear box in the laboratory at WES [112] indicated an angle of internal friction ϕ_s of 41° for a loose condition and 55° for light compaction; cohesion was not observed. For a very conservative angle of internal friction of 42°, the shear strength of the combined stone column and surrounding soil was less than that of the stone column for normal stresses greater than about 1600 psf (76.6 kN/m^2).

The field and laboratory tests just described indicate that the composite stone column-soil mass within the unit cell may not always develop the full shear strength of both materials when acting alone. Therefore, composite behavior may control stability analyses for conditions of very weak soils and/or strong stone columns (i.e., large angles of internal friction and/or large normal stresses). For soils having shear strengths as low as 100 psf (4.8 kN/m^2), the Japanese routinely use a stress concentration factor n of about 4 and an angle of internal friction ϕ_s of 30° (or more) for sand compaction piles. These numbers can be translated to stone columns and used as a lower bound for selecting stone column design parameters for weak soils. For comparison, stability analyses performed using $n = 4$ and $\phi_s = 30°$ give *very roughly* the same shear strength as using $n = 2$ and $\phi_s = 42°$. The latter parameters are sometimes used for the analysis of stone columns. These results suggest local bearing failure in weak soils can probably be avoided using for stone column design parameters equal to or less than about $n = 2$ and $\phi_s = 42°$; higher design values in very soft soils should not be used without further analysis (refer for example to Appendix B) or testing. Finally, this general discussion indicates that sand compaction piles are an attractive alternative to stone columns from the standpoint of both strength and economics for stability problems involving very soft and soft soils.

Shear Strength of Cohesive Soil

The shear strength measured in the field by vane testing should be multiplied by the correction factor μ originally proposed by Bjerrum [113]

TABLE 10. MEASURED ANGLE OF INTERNAL FRICTION OF STONE COLUMNS.

Location	Type Stone	Angle of Internal Friction, ϕ_s	Type Test	Comments
Jourdan Road Terminal, New Orleans, La.	Angular crushed limestone; 3/4 - 3.5 in. dia. D = 3.5'	$\phi_s = 54^\circ$ $\phi = 21^\circ$ (c = 260 psf)	Field - DS[1]; Double Steel ring; self reaction; DS - single steel ring; Reaction Column	Two steel rings were used to prevent local bearing failure (see Fig. 71); [71] Local bearing failure in soft clay; ϕ is effective ϕ of stone-soil system - not ϕ of stone
Santa Barbara, California	Gravel (some crushed); 3/4 - 3 in. dia. D = 3.5 ft.	$\phi_s = 38^\circ$ (c = 250 psf)	Field - DS; single steel ring; stone only	
		$\phi = 27^\circ$ c = 700 psf	Field - DS; single steel ring; DS; sheared stone column and tributary soil ($a_s = 0.36$)	For normal stress > 1540 psf → the shear strength mobilized in composite mass was less than for the stone column alone [30]
Steel Bayou, Miss.	Rounded gravel (GP) D = 2.8 ft.	$\phi = 33.1^\circ$ c = 425 psf (Saturated c = 122 psf)	Field - DS; single ring; reaction column	Measured strength > than strength for $\phi_s = 42^\circ$ and c = 0 when normal stress > 1600 psf; Soaked tests give lower ϕ; [111]
	No. 6 - 1-1/2 in. dia.; D = 3.0 ft.	$\phi_s = 55^\circ$ (c = 0) $\phi_s = 41^\circ$ (c = 0)	Lab DS - high density Lab DS - low density	Laboratory DS using a 24 in. by 24 in. shear box; high density = 107 pcf, low density = 90-100 pcf; Soaked tests had 4 - 7° lower ϕ; fine silty mat. adhering to stone may have affected ϕ soaked; continuous shear gave greater strength than step load [112]

1. DS = direct, quick shear test.
2. Unit Conversions: 1 ft. = 0.305 m; 1 in. = 25.4 mm; 1 psf = 47.9 N/m^2.

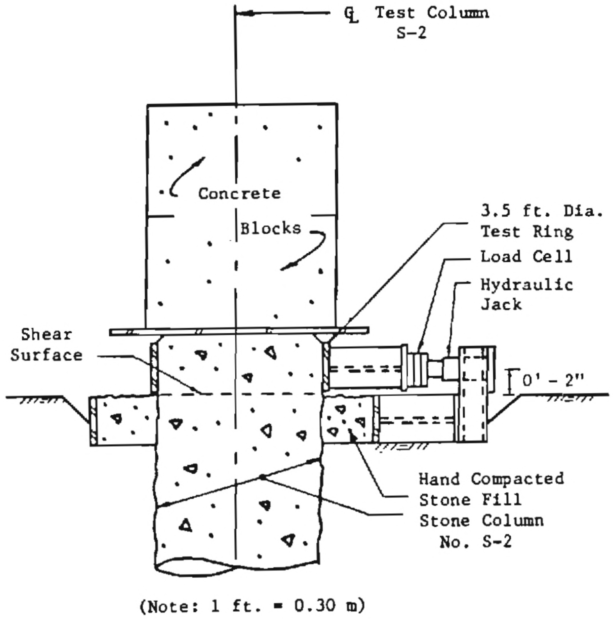

FIGURE 71. DOUBLE RING SHEAR TEST USED AT JOURDAN ROAD TERMINAL [71].

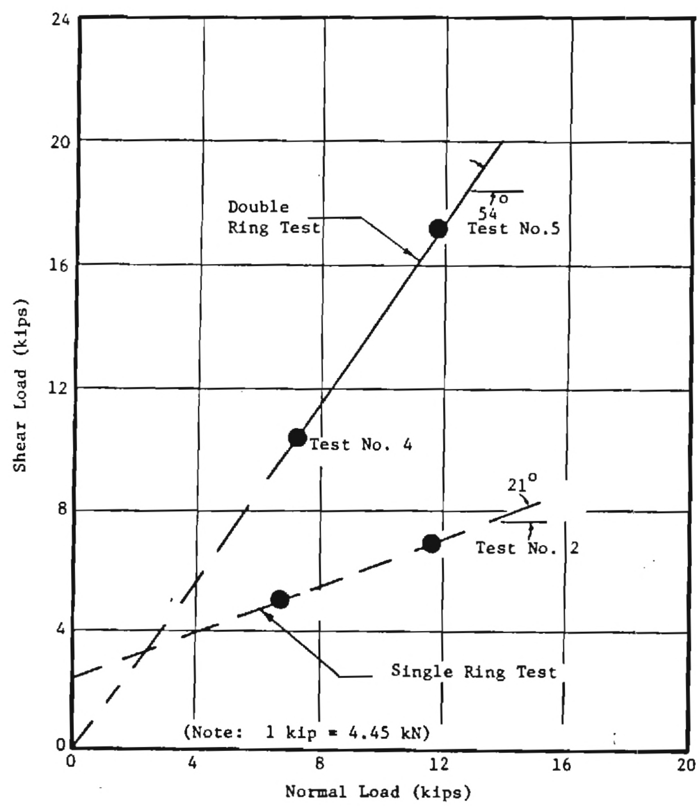

FIGURE 72. COMPARISON OF SINGLE AND DOUBLE RING LATERAL LOAD TEST RESULTS AT JOURDAN ROAD TERMINAL [71].

and shown in Fig. 73. In addition to the original data of Bjerrum, a number of additional data points compiled by Ladd [114] are included on the figure. All of these data points were obtained by back-calculation from full-scale embankment failures (without stone columns). Shear strength parameters obtained from field vane shear tests were used together where appropriate with circular arc stability analyses. Therefore, the factor μ applies an average correction to the vane shear strength that gives the net effect of all errors inherent in evaluating shear strength and performing the stability analyses. These errors have been discussed by Flaate and Preber [96] and later by Ladd and Foott [102].

Considering the scatter in data shown in Fig. 73, the possible variation in the correction factor μ is about ± 25 percent for plasticity indexes between 20 and 50. Stated in slightly different terms, this means that (for the cohesive soil) a ± 25 percent variation in the calculated safety factor can be expected in a real situation. To account for this possible variation, the vane shear test results should be analyzed as a group, and the lower 25 percentile of the shear strength used in design.

Design Parameters

The safety factor and values of the stress concentration factor n and ϕ_s used by several organizations are given in Table 11. Design values of n vary from 1.0 to 2.0 for stone columns, and the angles of internal friction from 40° to 45°. A miminum safety factor of 1.3 to 1.5 is recommended with respect to a general stability failure. For design a value of the angle of internal friction ϕ_s of the stone of no more than 42° to 45° is in general recommended for a good quality crushed stone; for gravels a value of 38° to 42° is recommended; these values of ϕ_s should be used with a stress concentration factor n of 2.0 (under some conditions a n of 2.5 might be used) until more field verification is developed. Where a high quality crushed stone is used, a ϕ_s of 42° can be employed for most applications with cohesive soils having shear strengths between 200 and 500 psf (10-24 kN/m^2). For soils having shear strengths less than about 200 psf (10 kN/m^2), a reduced ϕ_s may be prudent. For strengths greater than about 800 psf (38 kN/m^2), use of a ϕ_s of 45° is recommended. Each improvement application should be considered individually taking into account the possibility of a local bearing failure as described in Appendix B. Field direct shear tests, described subsequently, are also highly desirable in evaluating special applications.

FIELD LOAD TESTS

Field load tests are an important part of the overall design verification for stone columns just as conventional pile load tests are commonly used in practice. Load tests are performed to evaluate the (1) ultimate bearing capacity, (2) settlement characteristics, (3) shear strength of the stone column or the composite stone column-soil strength, and (4) to verify the adequacy of the overall construction process. The type and number of field tests performed depends upon the specific application of the stone

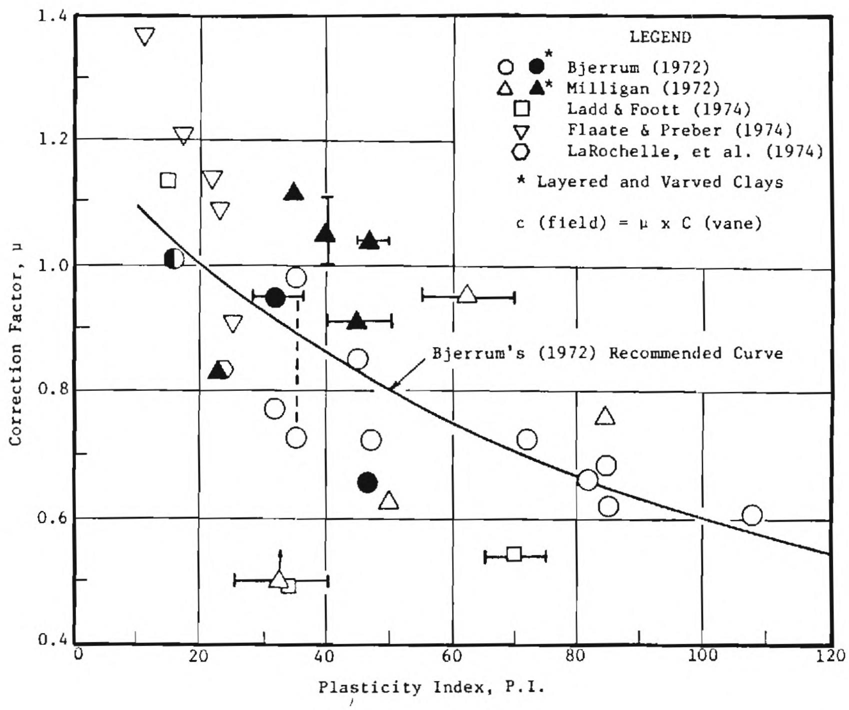

FIGURE 73. FIELD VANE CORRECTION FACTOR AS A FUNCTION OF PLASTICITY INDEX BACKCALCULATED FROM EMBANKMENT FAILURES [102].

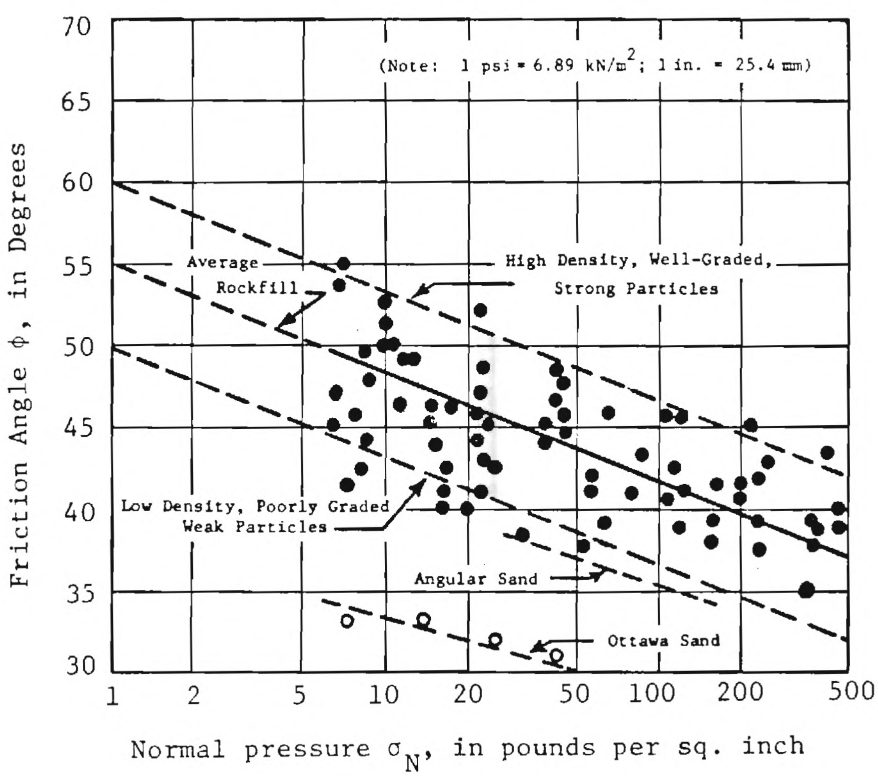

FIGURE 74. EFFECT OF NORMAL STRESS ON ANGLE OF INTERNAL FRICTION MEASURED IN LARGE TRIAXIAL CELL: ROCK FILL 3/4 TO 8 IN. MAX. SIZE (Adapted from Ref. 115).

TABLE 11. DESIGN PARAMETERS USED BY SELECTED ORGANIZATIONS FOR STABILITY ANALYSES OF STONE COLUMN REINFORCED GROUND.

Organization	Stress Concentration n	ϕ	S.F.
Vibroflotation Foundation Company	2.0	42^{o}	1.25 - 1.5
GKN Keller	2.0	45 (1) 40	1.3 - 1.4 (5)
PBQD (2)	1.0 - 2.0 (3)	42	1.3
Japanese (Sand compaction piles)	3 - 5 (4)	30 - 35 (4)	1.2 - 1.3

1. GKN Keller uses in Germany 45^{o} for crushed stone and 40^{o} for gravel.
2. Parsons, Brinkerhoff, Quade and Douglas, Inc. used these parameters at the Jourdan Road Terminal based on an instrumented Field Stability Test.
3. A stress concentration value n of 1.0 was used for all strata during construction and surcharge periods. A stress ratio of 2.0 was used after the surcharge period. A stress concentration factor of 3 to 5 was measured at the ground surface with the value increasing as consolidation occurred.
4. Higher values of stress concentration factor and angle of internal friction are also used in Japan for sand compaction piles.
5. German Codes generally require a minimum safety factor of 1.4.

columns, and also many other factors such as subsurface conditions and the degree of conservatism used in design.

Angle of Internal Friction of Stone Column

The angle of internal friction ϕ_s of the stone column, when measured in the field, should be evaluated using a double ring, direct shear test. A double ring shear test prevents a local bearing failure from occurring in front of the column which, as previously discussed, can indicate a shear strength less than that of the stone column.

Double ring direct shear tests were performed in the field at Jourdan Road Terminal on a 3.5 in. (90 mm) crushed stone, and in a large laboratory direct shear apparatus at WES on 1.5 in. (38 mm) densified gravel used at Steel Bayou. These tests indicated angles of internal friction ϕ_s above 50°. Extensive large diameter triaxial test results for rockfill have been summarized by Leps [115]. These results indicate that for average rockfill at moderate density, the angle of internal friction is above 45° for normal stresses less than about 20 psi (138 kN/m^2) as shown in Fig. 74. This figure also shows that the angle of internal friction decreases as the normal stress increases. Therefore, field direct shear test ϕ_s values, often obtained for low normal stresses, should be corrected to reflect the anticipated normal stress level. The average curve given in Fig. 74 can be used to correct field test results. For example, the ϕ_s value of 54° measured at Jourdan Road at an average normal stress of about 6.85 psi (47 kN/m^2) would be reduced to about 50° after correcting for a normal stress of 20 psi (138 kN/m^2).

The above results indicate when accepted construction practices are followed and a large, good quality crushed stone is used, a direct shear test performed in the field using an upper and lower shear box should give an angle of internal friction greater than the recommended design values which were equal to or less than 45°. Therefore, when a competent contractor constructs stone columns following accepted construction practices, performing a double ring direct shear test in the field in general would contribute little additional useful information; in most instances the expense of performing the test would not be justified. At the beginning of the project, the stone columns should be carefully examined for general appearance, gradation, and intrusion of sand and/or soft soil into the stone column. Several density tests are also recommended. If the stone column appears to be satisfactory, a double ring direct shear test is not in general recommended to evaluate just the angle of internal friction of the stone.

When required, the angle of internal friction ϕ_s can, as an alternative to field testing, be evaluated in the laboratory. A large triaxial apparatus is recommended since it probably more closely duplicates the less well-defined failure plane observed in a small-scale direct shear test in the laboratory (Fig. 75).

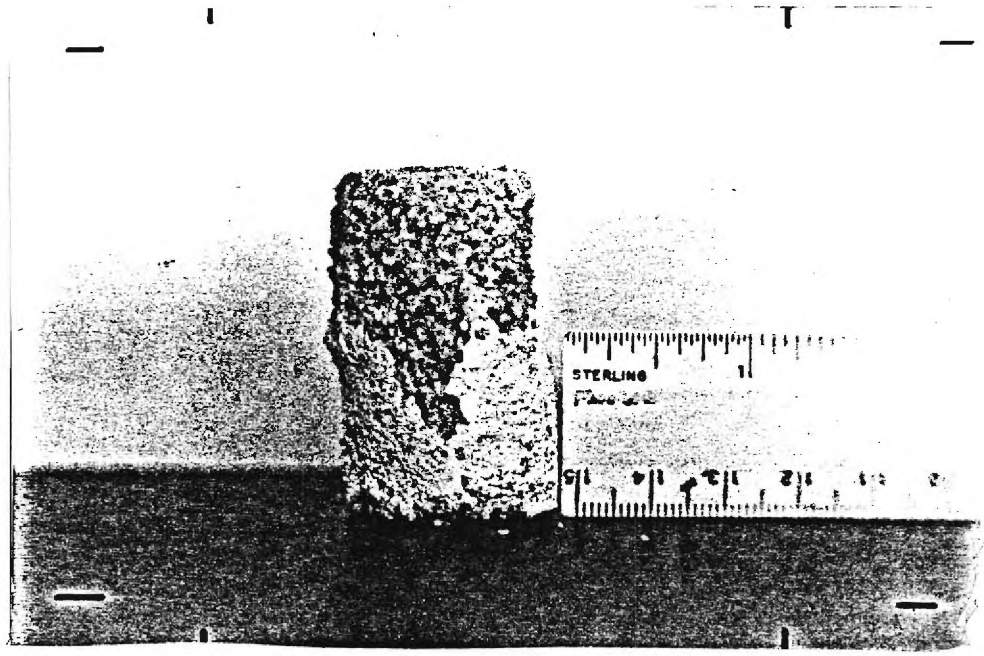

FIGURE 75. COLUMN AFTER FAILURE OF COMPOSITE MODEL STONE COLUMN - SOIL MASS TESTED IN MODIFIED DIRECT SHEAR APPARATUS.

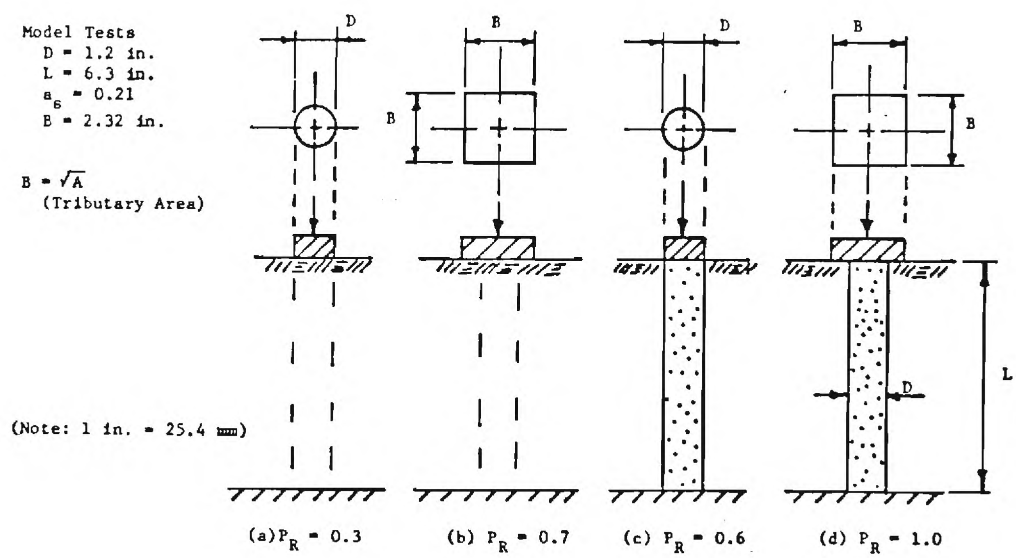

FIGURE 76. RELATIVE ULTIMATE LOAD CAPACITIES P_R FOR DIFFERENT TYPE LOADINGS WITH AND WITHOUT STONE COLUMNS: SMALL SCALE MODEL TESTS L/D = 6.3, a_s = 0.21.

Composite Shear Behavior

At the present time relatively little is known about the composite behavior of the stone column-soil unit. Therefore, for stability applications involving weak soils having shear stresses less than about 300 to 400 psf (14-19 kN/m^2), at least two double ring, direct shear tests should be performed in the field to assess the composite behavior of the stone column and its tributary soil. Composite tests would also be desirable for stiff cohesive soils to aid in determining the composite shear strength and hence if larger values of ϕ_s than those presently recommended are possible. A test set-up similar to that used at Jourdan Road Terminal should be used (Fig. 71). The rings, however, should be large enough to accommodate both the stone column and tributary soil (Fig. 69).

In performing a direct shear test, care should be exercised to prevent eccentricity of loading for both the shear and normal loads; the shear force should be applied along the failure plane to avoid tilting. Tilting was found to be a problem at Steel Bayou. Also, tilting was a problem during the first load test at Jourdan Road Terminal when a testing arrangement, different than that shown in Fig. 71, was used. Deflection measurements should be made using independently supported reference beams located sufficiently away from the stone columns as to not move during the load test. The vertical change in height of the material should be measured during shear testing using at least three dial indicators. Also vane shear tests should be performed to determine the shear strength of the cohesive soil both within and around the steel ring. Density tests would also be desirable.

Vertical Load Tests

At the present time, the available theories have not been fully verified for estimating either the settlement or the ultimate capacity of stone column reinforced ground. Further, some method of insuring good stone column performance is required (i.e., quality control) on all projects. On many projects where a conservative design load is used one or some combination of the following techniques can be employed: (1) careful field inspection, (2) recorded ammeter readings, and (3) plate load tests. On large, important projects, however, at least one or two vertical load tests to at least 1.5 times the design load should be performed to insure the column will not undergo a shear failure, and proper construction technique has been followed.

Ultimate Capacity. Short-term, rapid load tests are recommended to evaluate ultimate stone column capacity where a low safety factor is to be used with respect to a bearing capacity failure (SF ≤ 2.0). This is often the case for embankment design. The load test program should, when practical, be planned to permit testing to failure rather than going to 1.5 or 2.0 times the design load. In general more information would be obtained from testing a single column to failure than testing a group of two columns to 1.5 times the design load.

Model test studies indicate that the method of applying the loading influences the mode of failure and hence the ultimate capacity of a stone column. To simulate stress conditions and stone column confinement representative of that which will exist beneath a foundation or wide loaded area, the load should be applied through a rigid footing or plate. The loaded area should correspond to the area tributary to the stone column. In general, the loading should not be applied to just the area of the stone column; the effect of lateral confinement and soil strength should also be included in the test by using a larger plate. Approximate relative ultimate strengths for different loading conditions obtained from small-scale model tests in a soft clay are illustrated in Fig. 76. Guide specifications for performing rapid vertical load tests were given in Chapter V.

Settlement. Many potential stone column applications such as bridge bents and abutments limit the design settlement to relatively low levels. For such applications settlement considerations will generally restrict the design load per column to values well below ultimate. In cohesionless soils, the immediate settlement, which can be defined by a short-term load test, will be most important. In cohesive soils, however, primary and secondary settlements will be much larger than the immediate settlement. In cohesive soils, long-term load tests are therefore required to define settlement characteristics; rapid load tests would only indicate ultimate bearing capacity. Long-term load tests should be considered on projects where stone columns are used in cohesive soils to support, for example, bridge bents, approach fills, or other applications where settlement is important.

In general, dead loading is most practical for long-term tests. The design load should be left on long enough to achieve *at least* 80 to 90 percent of primary consolidation. At Hampton, Virginia 100 percent of the primary settlement was achieved in about 4 months; consolidation occurred even faster at Jourdan Road Terminal. In soft or organic clays, secondary compression movements should also be measured if time permits.

The load test should be performed using as many stone columns as possible; more stone columns will lead to a more reliable settlement estimate. Twenty-three stone columns, for example, were used beneath and immediately adjacent to the load at Hampton, Virginia [27]; the ground was stabilized with a total of 45 stone columns in the test area. At Jourdan Road Terminal, New Orleans [71] a group of 14 stone columns were used. A group of 7 stone columns gives full confinement to the inner-most column when constructed using an equalateral triangular pattern. Frequently due to cost, however, only small groups will be load tested. A group of three stone columns in a triangular pattern offers a practical compromise if a very small group is load tested. The geotechnical properties of the soils within the load test area should, of course, be carefully defined by both test borings, vane shear tests, and laboratory tests.

The results of the load tests should be theoretically analyzed to determine the in-situ compression characteristics of the soil when reinforced with stone columns. The performance of the actual reinforced ground should then be predicted using the back-calculated material properties and the settlement theory presented in Chapter III. Finally a similar load test would be quite desirable on a similar foundation whose underlying soil has not been reinforced with stone columns. Such a test would permit estimating the improvement due to the stone columns.

Proof Tests. Proof testing of production stone columns was briefly discussed in Chapter II. Proof testing consists of usually rapidly applying a load to selected production piles as primarily a quality control technique. The proof test may also be conducted over a longer period of time to allow primary consolidation settlement to occur. In the past a rigid plate has been often used just the size of the column. Proof testing offers an inexpensive method to insure uniformity of stone column construction, and a minimum level of performance. Reaction for the test is obtained by jacking against a portable test frame loaded with dead weights, or against a heavy piece of construction equipment such as a crane. Loads of 30 to 35 tons can be applied using the portable frame, and 15 to 20 tons using a crane as the reaction. Depending upon the type reaction and stone column design load used, the test load in some cases might be only one-half or less of the design load.

The proof test will be effective in establishing quality control and performance characteristics primarily within a depth of about 3 stone column diameters. Therefore, for short stone columns 15 to 20 ft. (5-6 m) in length, the proof test offers an inexpensive method of evaluating the quality of stone column construction. For long columns, the proof test offers an inexpensive method of insuring quality control in only the upper portion of the stone column.

For jobs requiring the construction of a large number of stone columns such as embankment support, a minimum of 2 proof tests per job should, in general, be performed in the absence of other load testing. One additional proof test should be performed on each additional 300 stone columns after the first 300. This recommendation is in accordance with usual practice in England. Hence, in general, a job utilizing 600 stone columns should have as a minimum 3 proof load tests if other load testing is not specified. The proof test should be performed using a portable frame following the recommendations given previously. Proof tests should be performed on suspect columns as indicated by visual observations and from examination of construction records.

BRIDGE AND RETAINING STRUCTURES

Non-Pile Supported Bridge Structures

Stone columns can be used to support interior bridge bents, integral end bent/abutments, and end bents on sloping earth abutments. Such applications for stone columns should in general be considered only for sites slightly marginal with respect to settlement, and requiring only relatively low levels of improvement. Settlement considerations would determine whether a given site is suitable for improvement with stone columns. In general, cohesive soils should be stiff, having shear strengths greater than about 1 ksf (50 kN/m^2). Stone columns should not be used for bridge bent support at sites underlain by deposits of peat.

In some areas slightly marginal loose to firm silty sands may be encountered having a silt content greater than about 15 percent. Such soils generally cannot be densified sufficiently using conventional vibroflotation techniques to permit the use of shallow foundations. Ground improvement using stone columns offers at such sites an excellent possible alternative to piles or drilled pier foundations.

Another potential use of stone columns is for the foundation support of short, single span bridge end bents or combined end bent/abutments. Single span bridges would be less affected by differential settlement than continuous multispan bridges, and could therefore withstand greater amounts of total settlement which would govern the design. This application in weaker soils would be particularly attractive on lower volume roads. From the standpoint of limiting settlement, potential sites for this application should generally be underlain by firm cohesive soil having a shear strength greater than about 600 to 800 psf (30-38 kN/m^2) or loose silty sands.

About 40 bridge abutments in England have been supported on stone columns [116]. Typically, the bridge is supported by a counter-fort wall and concrete footing constructed above the stone column as shown in Fig. 77. Frequently, the design criterion of these walls has been a maximum of about 1 in. (25 mm) of settlement. Stone columns in England have been used to improve slightly marginal sites having shear strengths greater than about 1 ksf (50 kN/m^2).

Pile Supported End Bents

To improve stability of the embankment or support, for example, Reinforced Earth abutment walls, it may be necessary to improve the ground using stone columns at sites underlain by weak soils. For such applications where end bents are pile supported, the piles should be driven before constructing the stone columns and reinforced earth wall. Past experience has shown that stone columns can be constructed within about 3 ft. (0.9 m) of an existing pile. This construction sequence will result in down-drag on the piles, which should be considered in design. Also, the stone column pattern and pile bent configuration should be laid out before construction at the same time.

For sites underlain by very soft to soft cohesive soils, large embankment settlements will often occur, particularly in organic soils, as a result of vertical consolidation and lateral spreading. Use of a safety factor of 1.5 with respect to rotational failure will not insure small settlements; at one site involving organic soils settlements up to 18 in. (460 mm) occurred even though a safety factor of 2.0 was used with respect to a rotational stability failure.

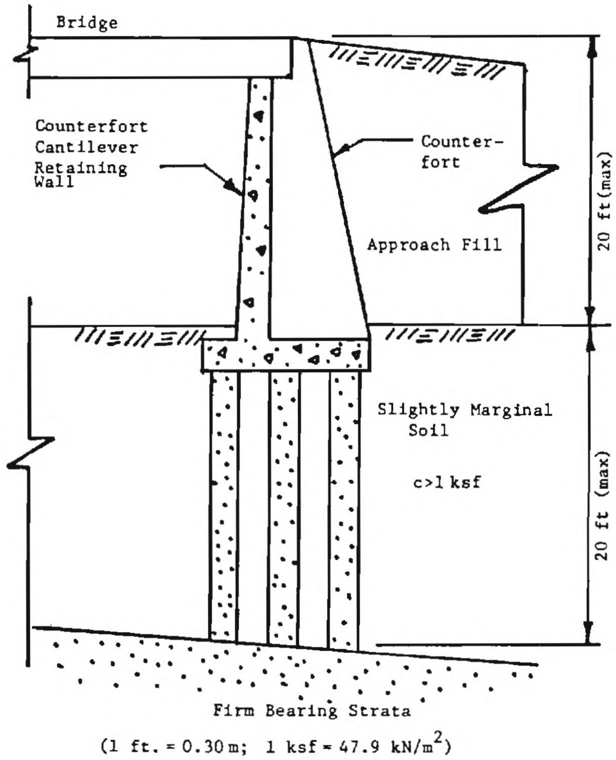

FIGURE 77. SUPPORT OF BRIDGE ABUTMENT ON STONE COLUMNS - ENGLAND.

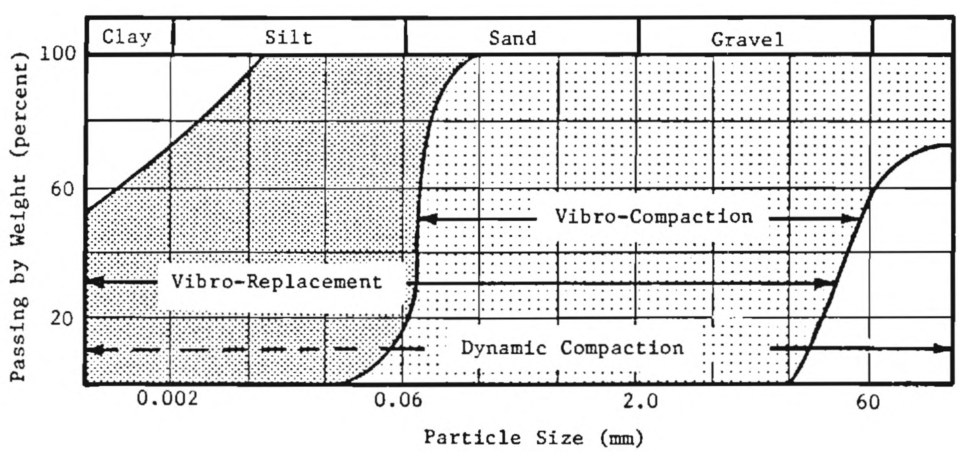

FIGURE 78. ALLOWABLE RANGE OF SOIL GRADATION FOR VARIOUS METHODS OF GROUND IMPROVEMENT [125].

Retaining Structures

Stone Column supported Reinforced Earth retaining structures have been used at Clark Fork, Idaho [10], Jourdan Road Terminal, New Orleans [71], and Rouen, France [63]. At Jourdan Road Terminal, a Reinforced Earth wall which was tested to failure underwent up to 1.6 ft. (0.5 m) of consolidation settlement without damage. At that time, the wall was forced to fail by surcharging and excavation in front. During failure the wall settled an average of 3.0 ft. (0.9 m). After failure, the Reinforced Earth wall panels were found to be in good condition, the embankment and wall having failed as a rigid block; a separation did occur at the center of the wall. Also, a Reinforced Earth wall caught in the middle of a landslide moved downward 16 ft. (5 m) and laterally 20 ft. (6 m) with little damage. The use of stone columns to support soil reinforced systems such as Reinforced Earth abutments or retaining walls, results in a very compatible, flexible construction. Undoubtedly stone column support of retaining structures (either conventional or Reinforced Earth) offers an important potential application of stone columns.

Retaining structures not carrying superstructure loads have been supported on stone column improved ground having shear strengths as low as 200 to 400 psf (10-20 kN/m^2). The resulting settlements, however, have been on the order of 1 to 2 ft. (0.3-0.6 m). Therefore, from the standpoint of settlement, for some applications stone columns would be limited to better soils.

Discussion

Highway engineers in the past have usually been reluctant to support bridge bents and abutments on shallow foundations. In the future, however, shallow foundations and the use of Reinforced Earth abutments will likely become more common due to economic considerations. The support of bridge bents for grade separations and bridge abutments on stone columns is a logical extension to the use of shallow foundations and the stone column technique. Use of stone columns beneath bridge bents would tend to reduce the amount of differential settlement between the embankment and bridge, which has always been a serious maintenance problem.

The bridge bent foundations must, of course, be designed to limit total and different settlement to tolerable levels. Bozozuk [44] has recently found, based on extensive field data, that conventional bridge foundations can safely undergo total vertical settlement up to 2 in. (50 mm); settlements greater than 6 in. (150 mm) result in damage. A more indepth study of bridge settlements has been presented by Moulton and Ganga Rao [127].

GENERAL DESIGN CONSIDERATIONS

Alternatives

Stone columns are, under certain conditions, a very useful ground improvement technique that should in the future be considered for many jobs as a potential design alternative. Stone columns, however, are certainly not necessarily either the most desirable or economical solution to many problems; they are merely another useful technique that should be carefully evaluated. All reasonable alternatives should be compared considering (1) overall performance, (2) level of reliability, and (3) total project cost including inspection, load tests, etc. Possible design alternatives in addition to stone columns that should be considered for embankment support include removal or displacement, stage construction, and/or preloading, bridge structures, and other site improvement methods. For bridge bent support at slightly marginal sites preloading, removal, other site improvement methods, piles (such as precast concrete, auger cast or steel), and drilled piers offer possible alternatives.

Stone columns, in general, are most economically attractive for sites requiring column lengths less than about 30 ft. (9 m), and preferably about 20 ft. (6 m) in length. The approximate cost of constructing stone columns (excluding the cost of stone) on a moderate size job involving more than about 8000 linear ft. (2400 m) of columns is about $8 to $10 per linear foot ($26-$33/m). For several sites, the cost of stone has been found to be approximately equal to the cost of constructing the column. Stone costs, however, are directly related to the distance to the source and can vary considerably. Therefore stone cost is an important item that must be considered separately for each potential application. Rigid stone columns would have approximately the same cost as conventional columns.

Environmental Considerations

Construction of stone columns using the conventional wet vibro-replacement process is a messy operation involving large quantities of excess silty water. The necessary steps should therefore be taken to prevent pollution as pointed out in Chapter V. Strict environmental regulations at some sites may even prohibit the use of water in constructing stone columns. At two such sites in Nova Scotia, Canada, for example, the dry process utilizing air has been used to construct stone columns. In England, the dry process is frequently used in developed areas because of environmental restrictions.

Design

Soil Gradation. Stone columns can be constructed by the vibro-replacement technique in a variety of soils varying from gravels and sands to silty sands, silts, and clays (Fig. 78). For embankment construction, the soils are generally soft to very soft, water deposited silts and clays. For bridge bent foundation support, silty sands having silt contents greater than about 15 percent and stiff clays are candidates for improvement with stone columns. In sands marginally unacceptable to vibroflotation, construction of stone columns not only replaces a portion of the silty sand with stone, but also improves to some extent the silty sand. About a dozen structures and tanks have already been supported on stone column reinforced silty sands and sandy silts at marginal sites within the United States [68].

Soil Shear Strength. Stone columns should not be considered for use in soils having shear strengths less than 150 psf (7 kN/m^2). Also stone columns in general should not be used in soils having sensitivities greater than about 5; experience is limited to this value of sensitivity [14]. Caution should be exercised in constructing stone columns in soils having average shear strengths less than about 400 psf (19 kN/m^2) as originally proposed by Thorburn [18]. In such soft to very soft soils hole collapse, construction technique, and interaction of the stone column and surrounding soil (composite action and local bearing failure) are important considerations. Intrusion of the soft soil into the voids of the stone, although of lesser concern, should still be considered in the very low strength soils.

At Jourdan Road Terminal, the shear strength of the upper 20 ft. (5.4 m) was on the order of 200 to 300 psf (9-14 kN/m^2). At Hampton, Virginia, the median value for the softer zones was about 380 psf (18 kN/m^2). The lowest two values observed at the site were 180 and 200 psf (8.6-9.6 kN/m^2), and constituted about 4 percent of the shear strength values in the poorer strata. These two examples serve as a guide to the strength of weak soils in which stone columns have been constructed.

For sites having shear strengths less than 350 to 400 psf (17 to 19 kN/m^2), use of sand for stability applications should be given consideration. Sand is often readily available and usually inexpensive compared to stone. Use of sand piles would, however, generally result in more settlement than for stone columns.

Stone Gradation. Typical stone column gradations used in the past were given in Chapters V and VI. The gradation selected for design should (1) follow a gradation that can be economically and readily supplied and (2) be coarse enough to settle out rapidly. In very soft soils intrusion of soil into the voids is also of some concern. Each specialty contractor prefers a different gradation, and has differing philosophies on handling special problems encountered during construction (refer to Chapters II and V). For soils having shear strengths greater than about 250 psf (12 kN/m^2), gradations similar to Alternate No. 1 or 3, Chapter VI, are recommended.

To reduce the possibility of intrusion, the gradation should be made finer with decreasing strength for very soft clays. For cohesive soils having strengths less than about 250 (12 kN/m^2), the finer side of alternate gradations No. 2, 3, or 4 (Chapter V) or an even finer gradation such as sand should be used. The use of a fine gradation such as sand would require a bottom feed system of column construction (Chapter II) or the use of sand compaction piles.

Ultimate Capacity. The theories presented in this report should be used as a general guide in estimating the ultimate capacity of stone columns. The selected design load, however, should take into full consideration site conditions, past experience and sound engineering judgement. Design loads applied to each stone column typically vary depending on site conditions from about 15 to about 60 tons. Table 12 gives typical design loads for foundation support where settlement is of concern; for stability problems such as embankments, the design loads can be increased. The specialty contractors who construct stone columns are a valuable source of technical support information concerning typical design loads, past experience, and potential problems for each specific site.

Area Replacement Ratio and Stone Column Pattern. Area replacement ratios used vary from 0.15 to 0.35; for most applications, the replacement ratio is greater than 0.20. Stone columns are usually constructed using the compact equilateral triangular pattern as compared to a square pattern (Fig. 13). Equilateral spacings used for stone columns vary from about 6 to 9 ft. (1.8-2.7 m), with typical values being 7 to 8 ft. (2.1-2.4 m). Spacings less than 5 ft. (1.5 m) are not in general recommended for the wet method.

Stone Column Diameter. The diameter of the constructed stone column depends primarily upon the type of soil present. The diameter of the column also varies to a lesser extent upon (1) the quantity and velocity of water used in advancing the hole and (2) the number of times the hole is flushed out by raising and dropping the vibroflot a short distance. Table 13 gives a preliminary guide for use in estimating the constructed diameter of a stone column in cohesive soils of varying shear strength.

Stone Density. The inplace density of the stone column effects both the estimated stone column diameter (refer to Chapter V) and also ϕ_s and hence the shear strength of the column. An inplace density of 120 to 125 pcf (18.8-19.6 kN/m^3) was measured at Santa Barbara, California [81]. The gradation of the constructed stone column, however, was significantly finer than the stone delivered to the site because of intrusion of local sand (refer to Table 6, Chapter VI). This sand increases the density of the stone column and would be expected to increase its shear strength. The local sand apparently comes from strata penetrated by the stone column during construction. For the same compactive effort required to achieve the observed field density, a density of 102 to 105 pcf (16.0-16.5 kN/m^3) was obtained for the stone using the gradation delivered to the site.

ASTM Test Method D-2049 can be used to establish the maximum and minimum relative densities of the stone used in stone column construction. A convenient alternative to D-2049 is ASTM C-29, which was developed for coarse concrete aggregate. Test C-29 is much simplier to run, but may give a slightly lower estimate of the maximum relative density than D-2049. ASTM C-29 test results for four selected stone column gradations are shown in Table 14. The dry densities shown in the table for these typical gradations vary from 92 to 109 pcf (14.4-17.1 kN/m^3).

TABLE 12. APPROXIMATE RANGE IN DESIGN LOADS USED IN PRACTICE FOR STONE COLUMNS.

Soil Type	Approximate Design Load (tons)	
	Fdn. Design(1)	Stability(1)
1. Cohesive Soil(2) $400 \text{ psf} \leq c \leq 600 \text{ psf}$	15-30	20-45
$600 \text{ psf} \leq c \leq 1000 \text{ psf}$	25-45	30-60
$c > 1000 \text{ psf}$	35-60	40-70
2. Cohesionless Soil	20-180 (see Note 1)	—

Notes: 1. In general, when stone column loads are given all the applied load is considered carried by the stone column.

2. Typical design loads for foundations on cohesive soils are 15 to 30 tons.

3. Unit Conversions: 1 psf = 47.9 N/m^2.

TABLE 13. APPROXIMATE RELATIONSHIP BETWEEN UNDRAINED SHEAR STRENGTH AND COMPLETED STONE COLUMN DIAMETER(1).

		Variation in Diameter			
Undrained Shear Strength (psf)	Typical Dia. (ft.)	Approximate Completed Stone Column Diameter (ft.)	Probe Dia. (in.)	Jetting Pressure (psi)	Comments
< 200	4 - 4.25	3.5	16 - 18	75 - 80	1 or 2 flushes
		3.5 - 4.0	18 - 19	75 - 80	1 or 2 flushes
		3.75 - 4.25	18 - 19	125 - 130	1 or 2 flushes
200 - 400	3.50 - 4.0	3.25	18 - 19	75 - 80	1 or 2 flushes
		3.5 - 3.75	18 - 19	125 - 130	1 or 2 flushes
		4.0	18 - 19	125 - 130	3 or 4 flushes
400 - 600	3.25 - 3.75	3.0	18 - 19	75 - 80	1 or 2 flushes
		3.25 - 3.5	18 - 19	125 - 130	1 or 2 flushes
		3.5 - 3.75	18 - 19	125 - 130	3 or 4 flushes
600 - 800 (2)	3.0 - 3.25	2.5 - 3.0	16 - 18	75 - 80	1 or 2 flushes
		2.75 - 3.0	18 - 19	125 - 130	1 or 2 flushes
		3.0 - 3.25	18 - 19	125 - 130	3 or 4 flushes
800 - 1000 (2)	2.25 - 3.0	2.25 - 3.0	18 - 20	125 - 130	1 or 2 flushes

Notes: 1. The hole diameter formed by jetting is less than the diameter of the completed stone column.
2. In firm to stiff soils the hole is sometimes augered at greatly increased expense to achieve the required diameter. Augering is sometimes done for slope stability applications.
3. Unit Conversions: 1 psf = 47.9 N/m^2; 1 ft. = 0.305 m; 1 psi = 6.89 kN/m^2.

TABLE 14. DRY DENSITY OF SELECTED STONE GRADATIONS FOR USE IN STONE COLUMNS.

Stone Gradation (1)	Density (pcf)		Void Ratio		75% Relative Density (pcf)	Comment (2)
	Loose	Dense	Max.	Min.		
Alternate 1	92	106	0.83	0.59	102	ASTM C-29 Test
Alternate 3	95	109	0.77	0.55	105	ASTM C-29 Test
Alternate 4	96	106	0.76	0.59	103	ASTM C-29 Test
Hampton [27]	96	108	0.73	0.56	105	ASTM C-29 Test

Notes: 1. Gradations are given for each alternate in Chapter V and for the Hampton stone in Table 6.
2. The stone tested had a saturated, surface dry specific gravity of 2.70.
3. Unit Conversion: 1 pcf = 0.157 kN/m^3.

Columns constructed using stone having the gradations shown in Table 14 would be expected to have densities varying between about 75 percent relative density (also shown in the table) and the maximum relative density. For the gradations tested, this density range is from 102 to 109 pcf (16.0-17.1 kN/m^3), with 105 pcf (16.5 kN/m^3) being a typical value. These densities agree very well with the 102 to 105 pcf (16.0-16.5 kN/m^3) obtained for the Santa Barbara stone (without sand intrusion).

Where native sands are present, a significant amount of intrusion of sand may occur into the stone column during construction. Therefore, the recommendation is made that the top of the stone column be carefully inspected after construction for intrusion of sand. Gradation and density tests should also be performed, particularly if the gradation appears to have changed. Admittedly, the density and gradation may be different at depth from that measured at the surface particularly when natural sands are present.

When sand intrusion is not observed, the stone can be assumed to have a dry density of about 105 pcf (16.5 kN/m^3) provided its gradation is similar to one of the gradations given in Table 14. Use of higher in-place dry densities would result in calculated stone column diameters being smaller than actually exist in the field. For stability analyses, the saturated unit weight of the stone should be used in calculating total stress below the groundwater table. The saturated unit weight of an open-graded stone is significantly greater than the dry weight. For example, a stone having a dry unit weight of 105 pcf (16.5 kN/m^3) has a saturated weight of 128 pcf (20.2 kN/m^3) if the specific gravity of the solids is 2.7.

Peat. Peat lenses are frequently encountered in soft compressible clay and silt deposits. A fibrous peat is preferable to a non-fibrous peat due to reinforcement given by the fibers. The presence of peat on several jobs has caused serious problems; refer to Chapter IV and VI for case histories involving peat. An adequate subsurface investigation must be performed to detect the presence of both peat and very soft zones.

In general peat layers greater in thickness than one stone column diameter should be avoided. Where peat is encountered, two or more vibrators can be attached together to give large diameter stone columns to satisfy this criterion. If peat lenses or layers are encountered thicker than one pile diameter, it may be feasible to use a rigid (concrete) column (which requires a special construction process) within the peat layer, and a stone column through the remainder of the strata (refer to Chapter II and VI).

Vibration. Construction of vibro-replacement stone columns causes some vibrations. A short distance from the vibrator, these vibrations are much less than the usually used maximum allowable peak particle velocity of 2 in./sec (51 mm/sec) as shown in Fig. 79.

Landslide Applications. The stone column theory and discussions presented

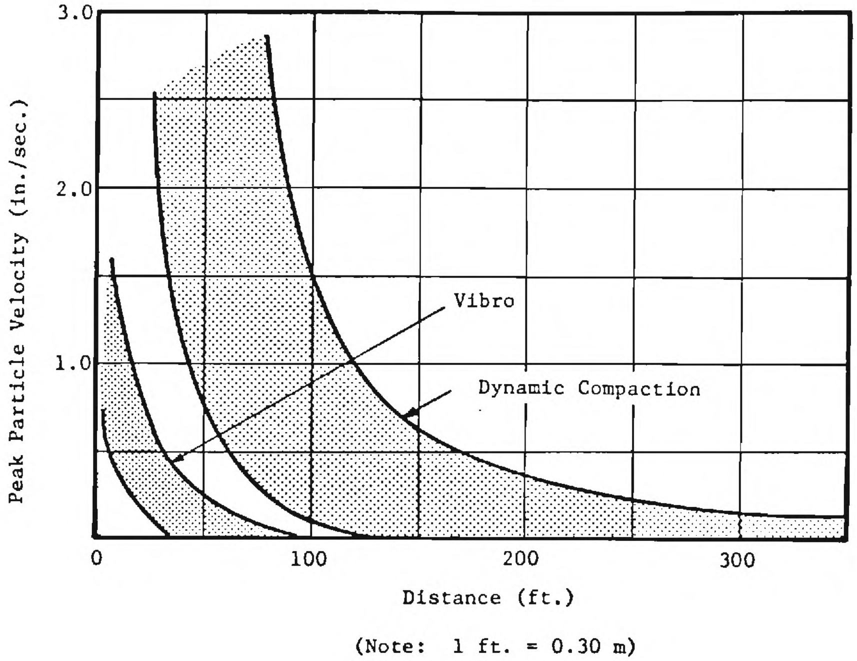

FIGURE 79. COMPARISON OF TYPICAL VIBRATION LEVELS INDUCED BY VIBRO AND DYNAMIC COMPACTION TECHNIQUES [125].

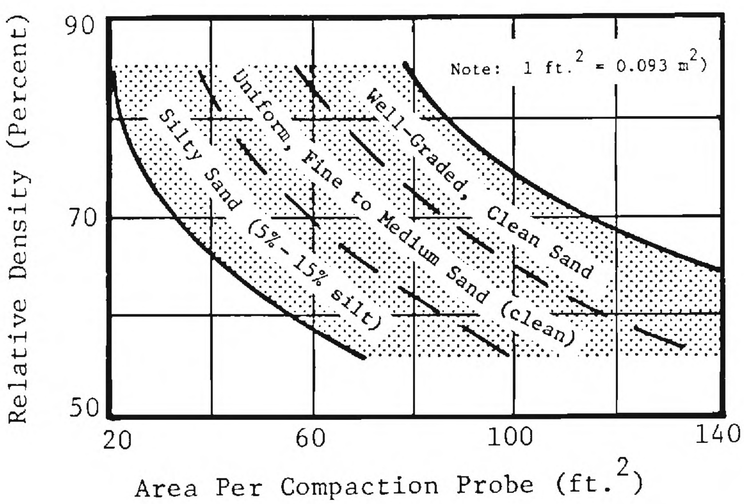

FIGURE 80. APPROXIMATE VARIATION OF RELATIVE DENSITY WITH TRIBUTARY AREA (Adapted from Ref. 125).

previously are applicable to landslide problems. In landslide applications getting sufficient normal stress on the stone columns to develop high shear resistance is sometimes a problem. A counterweight or berm can often be used to increase normal stress. Application of the berm also causes stress concentration in the column which further increases its effectiveness. Also, in problems involving landslide stabilization with stone columns, access to the area to be stabilized is often a problem. Finally, good field instrumentation is required to define in landslide problems the location and extent of the failure surface, and the role which water plays.

Liquefaction Applications. Stone columns have been used, for example, at Santa Barbara, California [30,31] and Kavala, Greece [126] to prevent liquefaction from occurring during strong motion earthquakes. Stone columns can take lateral earthquake loads (Chapter VI), if some support is provided surrounding the columns. Coarse stone has been found to perform better than sands with respect to liquefaction. The installation of stone columns also significantly increases the relative density of surrounding reasonably clean, loose sands that could liquefy. Fig. 80 can be used as a preliminary aid in selecting maximum tributary areas (and hence column spacing) to insure a certain minimum relative density in sands to be reinforced with stone columns. The installation of stone columns will also often increase the strength of silty sands and some cohesive soils, although several months or more may be required before the beneficial effect is observed.

Finally, stone columns act as drains helping to prevent a build-up in porewater pressure in cohesionless soils during an earthquake. Seed and Booker [128] have developed design curves for assessing the liquefaction potential of sands reinforced with stone columns. For most field conditions, water should flow essentially radially toward the stone column drain. Stone columns will act as ideal drains when the permeability of the drain is 200 or more times that of the soil [128]. For practical purposes, however, a permeability ratio of 50 is adequate. To insure vertical flow of water from the column, a permeable granular blanket should be placed over the stone columns on the surface.

Instrumentation. Finally, an important need exists for collecting additional information on stone column performance. Every available opportunity should be taken to install at least some field instrumentation and monitor performance both during and after construction. The subsurface conditions and geotechnical properties of the soils should be adequately defined and compiled in assessible reports. A discussion of desirable field instrumentation was given in Chapter IV.

CONCLUSIONS

Design methodology and specific design recommendations have been presented for predicting the ultimate capacity, settlement, and stability of ground improved using stone columns. The actual safety factor selected for a specific site should depend upon many factors including (1) how well the site conditions are known, (2) the degree of conservatism used in selecting material parameters, (3) whether the potential increase in shear strength of the clay due to consolidation was considered, and (4) the amount and quality of field control during construction. Although the methodology presented gives important guidance in stone column design, past experience, and sound engineering judgement must also be heavily relied upon. Specialty contractors are an important source of technical support and guidance in the design and construction of stone columns and should be consulted for each specific project.

Stone columns can be used to improve both soft cohesive soils and slightly marginal silty sands. In general, sites having peat layers greater in thickness than about one stone column diameter should be considered unsuitable for improvement using conventional stone columns. For thicker peat layers construction of a rigid (concrete) column through the peat and conventional stone column elsewhere is possible. Stone columns can be used to improve *slightly* marginal sites for the support of bridge bents. Use of stone columns for bridge support is not in general recommended if peat layers of any significant thickness are encountered.

When subjected to an external load, stress concentration in the stone column is a very important factor which accounts for a large part of the increases stability and reduced settlement of stone column improved ground. Measured stress concentration factors typically vary from 2 to 3 for stone column improved ground. Stress concentration depends upon a number of variables including relative stiffness of the stone column and tributary soil, applied stress level, and time. For very soft and soft cohesive soils, the interaction between the stone column and surrounding soil (composite action and local bearing failure) is also an important design consideration.

For some projects an accurate prediction of the rate of primary consolidation may be important to properly assess design alternatives. To reliably predict primary consolidation settlement rates, the permeability should be evaluated by field testing. Even then, observed settlement rates may be significantly different from that predicted, with the actual rate often being faster than predicted. For organic soils and many soft clays, secondary settlements may be important and should be considered in design.

CHAPTER VIII

CONCLUSIONS AND RECOMMENDED ADDITIONAL RESEARCH

STONE COLUMNS

The rapidly increasing cost of construction and numerous environmental constraints often placed on a project have greatly encouraged the in-situ improvement of marginal sites. Stone columns are one method of ground improvement that offers, under certain conditions, an alternative to conventional support methods in both weak cohesive soils and also loose silty sands. For each ground improvement problem, however, all feasible design alternatives must be thoroughly evaluated before selecting the most cost effective method.

Applications

Stone columns have been used for site improvement in Europe since the 1950's and in the U.S. since 1972. Stone columns have a wide range of potential applications. The following indicate a few of these applications:

1. Potential uses in highway construction include (a) embankment support over soft cohesive soils, (b) bridge approach fills, (c) bridge abutments, (d) widening and reconstruction work, (3) reduction in bridge length, (f) single span bridge support, (g) bridge bent and miscellaneous structural support.

2. Important applications of stone columns also exist for landslide stabilization and liquefaction problems involving bridge foundation and embankment support during earthquakes.

3. The use of stone columns for the support of bridge bent foundations and similar structures should in general be limited to slightly marginal sites. Such sites are defined as those where shallow foundations could be used without significant ground improvement except for slightly excessive settlements. For bridge bent foundations cohesive soils in general should have shear strengths greater than about 1 ksf (50 kN/m^2). Silty sands having silt contents too great to be improved using vibroflotation, can also be improved with stone columns for bridge bent support. For bridge bent support these silty sands should in general be loose to firm; silt contents would be greater than 15 percent.

4. The support of a Reinforced Earth retaining wall or abutment on stone columns gives a very flexible, compatible type construction capable of withstanding relatively large movements. Reinforced

earth walls have been supported on cohesive soils having shear strengths as low as 200 to 400 psf (10-20 kN/m^2). For these very soft to soft soils, wall settlement has been on the order of 1 to 2 ft. (0.3-0.6 m).

Stone Column Construction

Construction of stone columns was considered in detail in Chapters II, V, and VI. Stone columns are usually constructed using a vibrating probe often called a Vibroflot or Poker. Lateral vibration at the end of the probe is caused by rotating eccentric weights within the body of the probe. The eccentric weights are rotated using either electric or hydraulic power. Usually a fixed frequency vibrator is used operating at a frequency of about 1800 or 3000 rpm depending upon the specialty contractor.

In the wet process, the vibrator opens a hole by jetting using large quantities of water under high pressure. In the dry process, which may utilize air, the probe displaces the native soil laterally as it is advanced into the ground. Only the wet process has been used to date in the U.S. Because of the use of large quantities of water in the wet process, caution must be exercised to control from the environmental standpoint the water and silt from the construction process. The dry process is used primarily for environmental reasons and has been used in both Europe and Canada. Rammed stone columns are also sometimes used primarily in Belgium and India.

Inspection

Field inspection of stone columns is even more important than for conventional shallow or deep foundations. Important aspects of the vibro-replacement (wet) process requiring special attention during construction include (1) using a large quantity of water (about 3,000 to 4,000 gal/hr., 10-14 m^3/hr. average) at all times to maintain a stable hole and give a clean column, (2) in soft soils leaving the probe in the hole at all times with the jets running, (3) constructing the stone column in lifts no greater than 4 ft. (1.2 m), and (4) to insure good densification, repenetrating each new lift with the vibrator several times and also achieving the required ammeter reading. Rapid construction using the wet process is important in silts, sensitive clays, and peat. The discovery during construction of any peat layers should be brought to the immediate attention of both the project and design engineers. Finally, detailed construction records should be kept and analyzed for changes in quantity of stone consumption and time to both jet the hole and form the stone column.

Subsurface Investigation and Testing

A thorough subsurface investigation and evaluation of geotechnical properties are essential for the design of stone columns and the selection of the most suitable design alternative. The potential for use of stone columns and other possible design alternatives should be identified as early as possible during the subsurface investigation so that the exploration and testing program can be tailored to the specific design alternatives.

For sites underlain by firm to soft cohesive soils, use of field vane shear testing is recommended in the subsurface investigation. If either densification or stone columns are being considered as an alternative for improving silty sands, a sufficient number of washed grain size tests should be performed to accurately define the variation in silt content. Care in the subsurface investigation should also be taken to identify organic and peat layers.

The horizontal permeability of soft cohesive soils may be greater than 3 to 5 times the vertical permeability. Consolidation tests on horizontally orientated specimens cannot be used to evaluate the horizontal coefficient of consolidation (or permeability) of an anisotropic soil. Field pumping tests should be performed where a reliable estimate of the time rate of settlement is required for the success of the project, or for reliable comparisons of different design alternatives. To minimize smear effects, well points and wells should be installed by jetting if the vibro-replacement method of stone column construction is to be used. On routine projects laboratory permeability tests on vertical and horizontal samples can be used to evaluate the consolidation characteristics. The evaluation of the permeability (and hence consolidation characteristics) of a stratum is at best difficult to both perform and interpret; a high degree of accuracy of the estimated rate of primary consolidation settlement should therefore not be expected.

Stone Column Design

Stone column design to a large extent is still empirical, and past experience and practice plays an important role in design. Stone column design theories were given in Chapter III and design recommendations in Chapter VII. Specialty contractors are also an important source of technical support and guidance in the design and construction of stone columns, and should be consulted for each project. Specific conclusions concerning the design of stone columns are as follows:

1. The design load of stone columns is generally between 15 and 60 tons per column. For economic reasons, the thickness of the strata to be improved should in general be no greater than 30 ft. (9 m) and preferably about 20 ft. (6 m). Usually, the weak layer should be underlain by a competent bearing stratum to realize optimum utility and economy.

2. Caution should be exercised in the design and construction of stone column supported embankments or other structures on cohesive soils having average shear strengths less than 400 psf (19 kN/m^2); use of stone columns in soils having shear strengths less than 150 psf (7 kN/m^2) is not recommended. Also, construction of stone columns in soils having sensitivities greater than 5 is not recommended.

3. For embankment support in cohesive soils having a shear strength less than about 400 psf (19 kN/m^2), consideration should be given

to using sand as an alternative to the large stone traditionally used in stone columns. Sand is often readily available near the site, and frequently is considerably less expensive than crushed stone which may have to be imported from a considerable distance. Either bottom feed stone column equipment or sand compaction pile equipment can be used to construct sand piles. Sand compaction piles are routinely used in Japan for embankment support in cohesive soils having shear strengths as low as 100 psf (5 kN/m^2).

4. Conventional stone columns should not be used at sites having peat layers greater in thickness than 1 stone column diameter. A fibrous peat gives better support to a stone column than a non-fibrous peat. Rigid stone columns offer one solution to construction of stone columns in soils having peat layers or lenses. Two or more conventional vibrators can also be attached together to form a large diameter stone column to reduce the thickness to diameter ratio through the peat layer.

5. Stone columns act as drains and under favorable conditions can significantly decrease the time for primary consolidation to occur. Because of rapid consolidation settlement secondary settlement becomes a more important consideration when stone columns are used. Finally, the columns reduce the build-up in pore pressure in granular layers during an earthquake, and hence decrease liquefaction potential.

6. In general, a stress concentration factor n of 2 to 2.5 and angle of internal friction ϕ_s of the stone column of 38 to 45° should be used in theoretical analyses.

ADDITIONAL RESEARCH

Field performance information for stone column improved ground is needed for future design, and to develop a better understanding of the mechanistic behavior of stone columns. Some of the more important aspects of behavior needed from an applied viewpoint are as follows:

1. Improvement Factor. The actual reduction in settlement which is achieved when soft ground is reinforced with stone columns has not been well documented in the field. Full-scale embankment or group load tests need to be performed for varying soil conditions to establish the amount of improvement in terms of reduction in immediate and consolidation settlement. To develop improvement factors, settlement tests must be performed on both the unimproved and the stone column improved ground. These results should be used to verify existing theories for predicting settlement of stone column improved ground. Inductive coils (or other devices) should be used to measure the settlement of each compressible layer. To properly interpret the results, a thorough subsurface investigation should be made at each test site.

If a long-term load test is performed to evaluate settlement in stone column reinforced ground, a similar load test should also be performed on the unimproved ground to permit calculating the improvement factor. With careful planning, it may even be possible to evaluate for an embankment the reduction in settlement in areas improved with stone columns compared to unimproved areas. For example, lower fills may not require ground improvement compared to higher fills underlain by similar soils. Construction could be planned whereas primary consolidation is allowed to occur under similar fill heights in each area to indicate the amount of ground improvement.

2. Test Embankment Failure/Composite Behavior. For certain conditions of stress level and soil and stone column strength, the composite strength of the stone column and tributary soil can be less than that of the individual materials (refer to the discussion on stability in Chapter VII). To investigate composite type failures and overall strength of the composite mass, a section of an embankment should be constructed over a cohesive soil having a shear strength in the range of about 200 to 300 psf (10-17 kN/m^2). The embankment height should be increased until failure occurs to evaluate the actual beneficial effect on stability of improving the ground with stone columns. The actual failure surface should be accurately defined using a sufficient number of inclinometers. Double ring direct shear tests should be performed on the composite soil-stone column mass as discussed in Chapter VII. In general, the occurrence, effect and prediction of local bearing failures within stone column groups should also be studied in both the field and laboratory.

3. Stress Concentration and Stress Distribution. Use of stone columns for embankment stability problems will in the future continue to be an important application. Development of an economical design is dependent upon the use of realistic values of both stress concentration and angle of internal friction of the stone column. Both these factors are dependent upon a complex interaction between the stone column, soil, and embankment.

   In both prototype and test embankments, pressure cells should be placed in the stone column and soil at the embankment interface. Pressure cells could also be placed at several levels beneath the surface to develop important information concerning the variation of stress distribution and stress concentration with depth and time. Both Vautrain [63] and the Japanese [24] have performed such field measurements. Field measurements could be nicely supplemented by finite element analyses to study stress concentration, stress distribution, and the effects of lateral spreading.

4. General. A description of field instrumentation for specialized research projects is beyond the scope of this discussion. The above discussion does, however, point out some response information that is quite badly needed to better utilize stone columns. With the exception of intentionally inducing an embankment failure, this

data can be obtained by monitoring routine stone column projects. Field instrumentation can, of course, be used to help answer many other questions involving stone column behavior.

Important remaining unanswered questions that can be studied by a combination of full-scale field tests, model studies, and finite element studies include: (1) performance of stone columns not carried to end bearing, (2) stress distribution in large and small stone column groups, (3) effect of lateral soil movement on the settlement and general performance of both small and large stone column groups, (4) effect of method of construction, lateral stress, remolding and smear during stone column construction, and finally (5) interface slip and compatibility between stone column and ground settlement. Considerable additional research is needed to improve existing design methods and develop a complete understanding of the mechanics of stone column behavior. Every opportunity should certainly be taken to instrument stone column projects.

Lastly, an important need exists for a carefully planned field study to establish the effects of vibrator characteristics (such as horsepower, ampere draw, free vibration amplitude, operating frequency, and centrifugal force) on stone column performance. Also, a comparison of the performance of vibro-replacement stone columns, vibro sand columns (constructed using a bottom-feed system), sand compaction piles, and rammed stone and/or sand columns would add valuable information needed in selecting the most cost-effective ground improvement method for each site.

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APPENDIX A

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Kencho, Inc. 25030 Viking St. Hayward, California 94545 (Sand Compaction Piles) Note: Toyomenka (America) Inc. were formerly the trading company for the Kensetsu Kikai Chosa Co., Ltd. Vibrators. Kencho, Inc., a division of Kensetsu Kikai Chosa Co., Inc., is now selling its own equipment in the U.S. Phone: 415/887-3836	Mr. K. Hayashi Mr. Tony Sullivan Mr. Howard West

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APPENDIX B

LOCAL BEARING FAILURE OF AN ISOLATED STONE COLUMN

Stone columns are an effective method for resisting rotational shear failures involving soft clays in embankments and slopes [9]. For a conventional slope stability analysis, the resisting shear force F developed by the stone column is determined by multiplying the effective normal force, $\bar{W}_N$ acting on the shear surface by the tangent of the angle of internal friction of the stone, $\tan\phi_s$. The shear capacity, F, of the stone column can, under unfavorable conditions, be limited by a local bearing failure [129] of the stone column and cohesive soil behind the column as illustrated in Figs. 81 and 82.

Now consider the behavior of an isolated, single stone column surrounded by a cohesive soil. If the shear force in the stone column is sufficiently large compared to the strength of the surrounding cohesive soil, a secondary failure surface can develop in the stone column extending downward from the circular arc failure surface (Fig. 81). The resulting wedge of failed stone is bounded above by the circular arc failure surface. The lower failure surface develops within the stone at an angle resulting in the minimum resistance to sliding as defined by force F. The shear force, F, applied to the top causes the wedge (Fig. 82) to slide downward and laterally in the direction of movement of the unstable soil mass above. Sliding of the wedge of stone is resisted by the frictional resistance of the stone developed along the bottom of the wedge and the passive lateral resistance of the adjacent clay. If the passive resistance of the clay is not sufficient, the stone wedge undergoes a local bearing failure by punching into the clay. If a local bearing failure of the clay occurs behind the stone column, the capacity of the column is limited by the secondary wedge failure. A local bearing failure of the clay behind the stone column has been observed by Goughnour [129] during a direct shear test performed in the field on a stone column. Reduced strength of the composite mass was also indicated at Santa Barbara [30] and Steel Bayou [111].

Local Bearing Failure

The limiting shear force that can be applied if a bearing failure controls can be obtained for an isolated column by considering the equilibrium of the wedge shown in Fig. 81. This wedge together with the forces acting on it are illustrated in Fig. 82. The notation shown in this figure is used in the subsequent derivations and is as follows:

W_s = effective force of stone in the wedge

$\bar{\gamma}_s$ = effective (bouyant) unit weight of stone in wedge

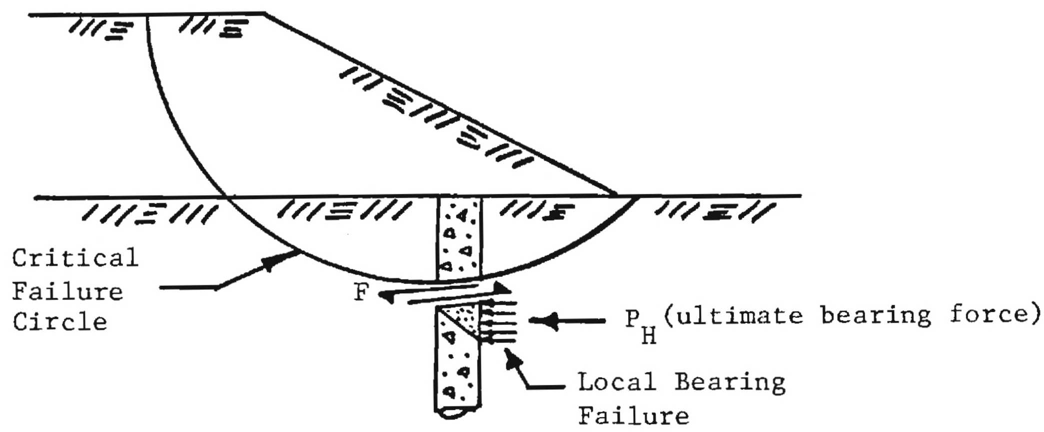

FIGURE 81. WEDGE TYPE LOCAL BEARING FAILURE OF A STONE COLUMN.

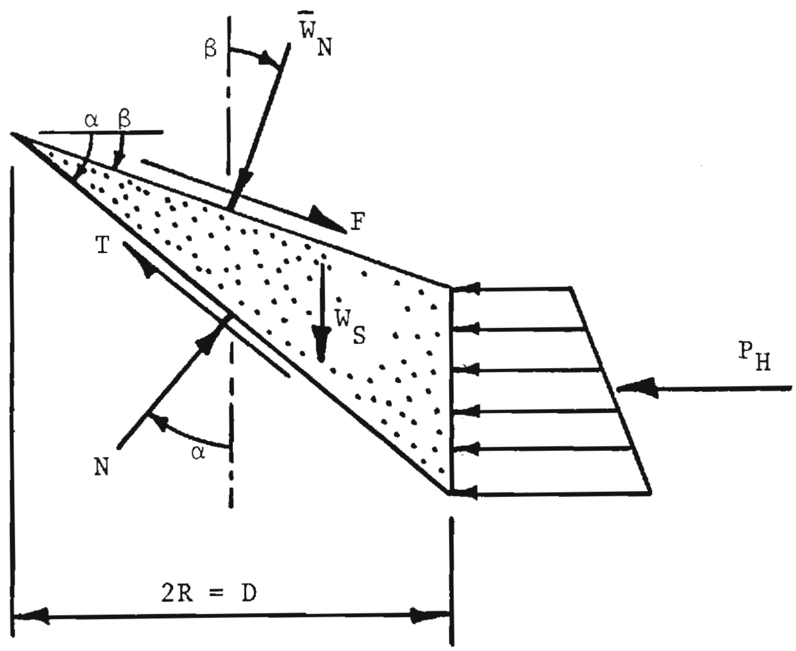

FIGURE 82. LOCAL BEARING CAPACITY FAILURE WEDGE IN STONE COLUMN.

P_H = ultimate lateral resistance of the clay acting on the wedge

N,T = normal and shear force, respectively, exerted on the bottom surface of the wedge

$\overline{W}_N$,F = normal and shear force, respectively, exerted on the top surface of the wedge

R = radius of the stone column

D = diameter of the stone column

ϕ_s = angle of internal friction of the stone

α,β = angle of inclinations of the lower and upper surfaces of the wedge, respectively.

The upper surface of the wedge makes an angle β with the horizontal.[1] This upper surface coincides with the circular arc failure surface (Fig. 81). The lower surface of the wedge makes an angle of α with the horizontal. Now consider equilibrium of the wedge. To develop the required relationship for F, first sum forces acting on the wedge in the vertical direction and solve for the unknown normal force N acting on the bottom of the wedge obtaining

$$N = \frac{W_s + \overline{W}_N \cos\beta + F \sin\beta}{\cos\alpha + \tan\phi_s \sin\alpha} \tag{53}$$

where the forces and angles are shown in Fig. 82.

Now sum the forces acting on the wedge in the horizontal direction, substitute for the unknown force N using equation (53), and solve for the limiting force F obtaining

$$F = \frac{\overline{W}_N(\sin\beta + \lambda \cos\beta) + \lambda W_s + P_H}{\cos\beta - \lambda \sin\beta} \tag{54}$$

where: $$\lambda = \frac{\tan\phi_s \cos\alpha - \sin\alpha}{\cos\alpha + \tan\phi_s \sin\alpha}$$

$$W_s = \pi(\tan\alpha - \tan\beta)R^3\overline{\gamma}_s$$

In the derivation of equation (54), the effects of adjacent stone columns and outward, lateral spreading of the stone columns were neglected. Neglecting the effect of adjacent columns should introduce a factor of conservation in predicting the effect of a local bearing failure [130-132]. These effects are offset by neglecting lateral spreading which should be on the unconservative side.

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1. R. R. Goughnour of the Vibroflotation Foundation Company has previously developed a solution similar in concept for the special case of $\beta = 0$. His solution handled lateral pressure on the column slightly differently than this solution.

Lateral Bearing Failure in Cohesive Soil

The ultimate passive pressure developed by the cohesive soil as the wedge pushes against it can be calculated using the theory presented by Broms [130] for a single, laterally loaded pile embedded in a frictionless soil. As shown in Fig. 83, the ultimate lateral pressure q_h at the surface is taken to be $q_h = 2c$ with the resistance increasing linearly over a depth of 3 pile diameters where it reaches a maximum limiting value of $q_h = 9c$. The total depth beneath the surface $h + z_o$ (Fig. 84) is considered in determining the 3 pile diameters. Near the surface, the failure occurs due to the upward flow of cohesive soil toward the surface. With increasing depth the failure becomes one of the plastic flow of the soil from the front of the pile around the sides (Fig. 83).

For a single, rough pile having full cohesion, plastic theory [130,131] indicates below a depth of approximately 3 diameters the ultimate lateral capacity is about $q_h = 11$ to $12c$. Use of an ultimate resistance of 9c, however, is felt to be prudent although it may be slightly on the conservative side. Further, the use of $q_h = 9c$ is reasonable since it is equal to the end bearing capacity of deep piles embedded in a cohesive soil. The value of $q_h = 2c$ used at the surface is also realistic since it equals about 40 percent of the bearing capacity of the clay in the vertical direction.

Now consider the ultimate lateral pressure developed on a wedge of stone making an angle α and β with the horizontal as shown in Fig. 82. Using the pressure distribution shown in Fig. 83, the ultimate passive pressure developed in the clay for a depth $(h + z_o) \leq 3D$ as illustrated in Fig. 84 is

$$P_H = \frac{14}{3} R\ c\ \psi\ [h + z_0 + R(1.714 + \tan\alpha)] \tag{55}$$

and for a depth $h + z_0 > 3D$:

$$P_H = 36R^2\ c\ \psi \tag{56}$$

where:
- R = radius of stone column
- c = cohesion
- ψ = $\tan\alpha - \tan\beta$
- h = depth of fill above the stone column
- z_0 = depth of the circular arc failure surface below the top of the stone

The sign convention used for α and β is shown in Fig. 84. Once a trial circular arc failure surface has been selected, the value of β is known. The angle α is then determined to give the minimum value of shear force F that can be applied to the top of the wedge before a bearing failure occurs.

Calculation of Limiting Shear Force

The limiting shear force F in each column for a given circular arc sliding surface is calculated as follows:

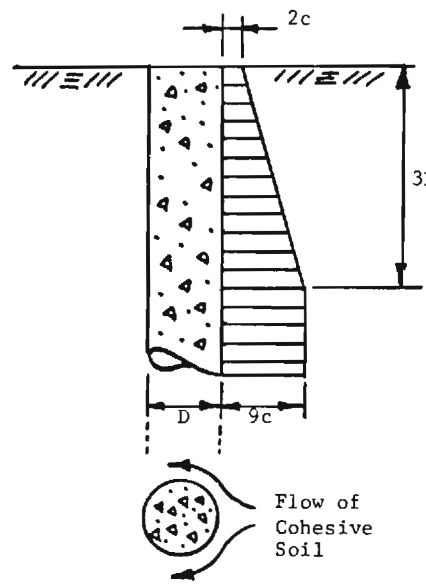

FIGURE 83. BEARING CAPACITY OF A RIGID PILE TRANSLATING LATERALLY IN A COHESIVE SOIL.

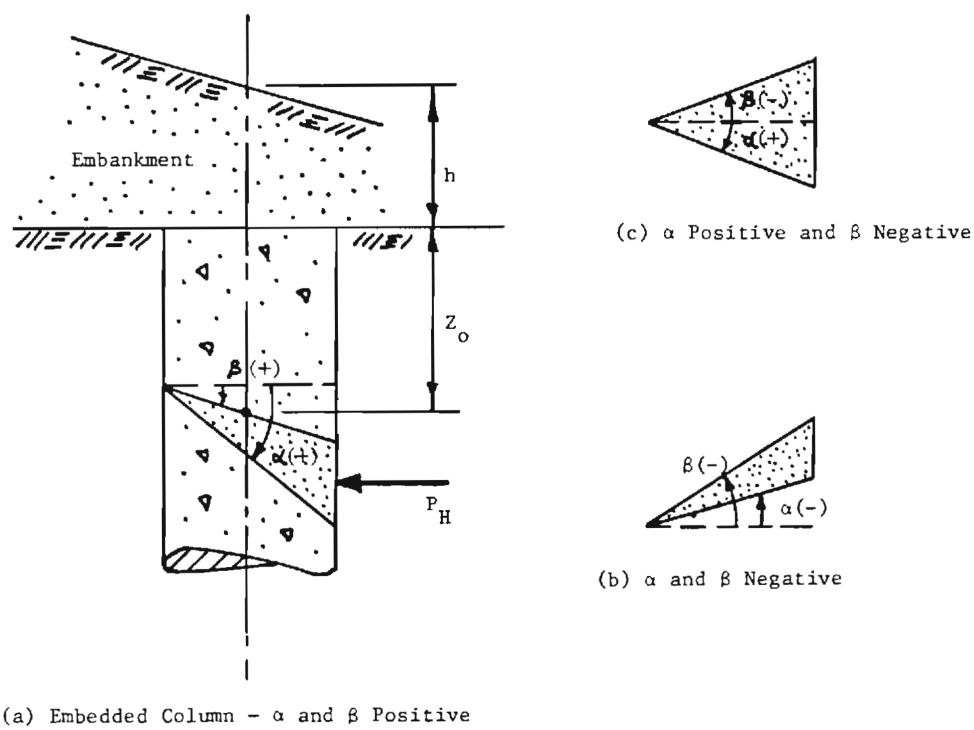

FIGURE 84. NOTATION USED IN FORMULAS FOR LOCAL BEARING FAILURE OF A STONE COLUMN.

1. Determine the angle β for a critical circle and calculate the effective normal force, $\overline{W}_N$ (Fig. 84) at the point on the stone column where the circular arc intersects the center of the stone column (Fig. 81).

2. Select at least three trial values of the angle of inclination α of the lower surface of the wedge.

3. For each value of α calculate the ultimate lateral soil resistance, P_H using equation (55) or (56) and a representative value of the undrained shear strength c of the cohesive soil.

4. For each value of α, calculate F for a bearing failure in the cohesive soil using equation (54).

5. Plot the shear force F obtained from equation (54) as a function of α and select the minimum value of F.

6. Calculate the shear force F that can act on the column if a local bearing failure does not develop: $F = \overline{W}_N \tan\phi_s$.

7. If a local bearing failure of the clay controls the force calculated in Step 5 will be less than that calculated in Step 6. In the stability analysis use the smaller of these forces (or reduce the value of ϕ_s used in design).

8. Repeat the analysis for several selected points along the failure surface.

Design Charts

Figures 85 through 95 present graphically the solution for local bearing failure of a single, isolated stone column for selected design parameters. The procedure for using the charts is as follows:

1. Select tentative design parameters and perform a stability analysis for the stone column improved ground. Plot the critical circle through the stone columns. Examine for the possibility of local failure several points along the critical circle where it intersects the center of the columns. Measure the inclination β of the circle (with the correct sign), and the depth $h + z_o$ of each point (Fig. 84).

2. Calculate the effective vertical force $\overline{W}_v$ acting on the stone column at the depth under consideration by multiplying the vertical effective stress times the area of the stone column. First calculate the effective body stress due to the stone column at the selected point. Use the bouyant unit weight of the stone below the groundwater table. Then calculate the vertical stress σ due to the embankment above the stone column and obtain the stress concentration in the column using $\sigma_s = \mu_s\sigma$ (equation 8b). Add the body stress to σ_s and multiply by the area of the stone column to obtain the effective vertical force $\overline{W}_v$.

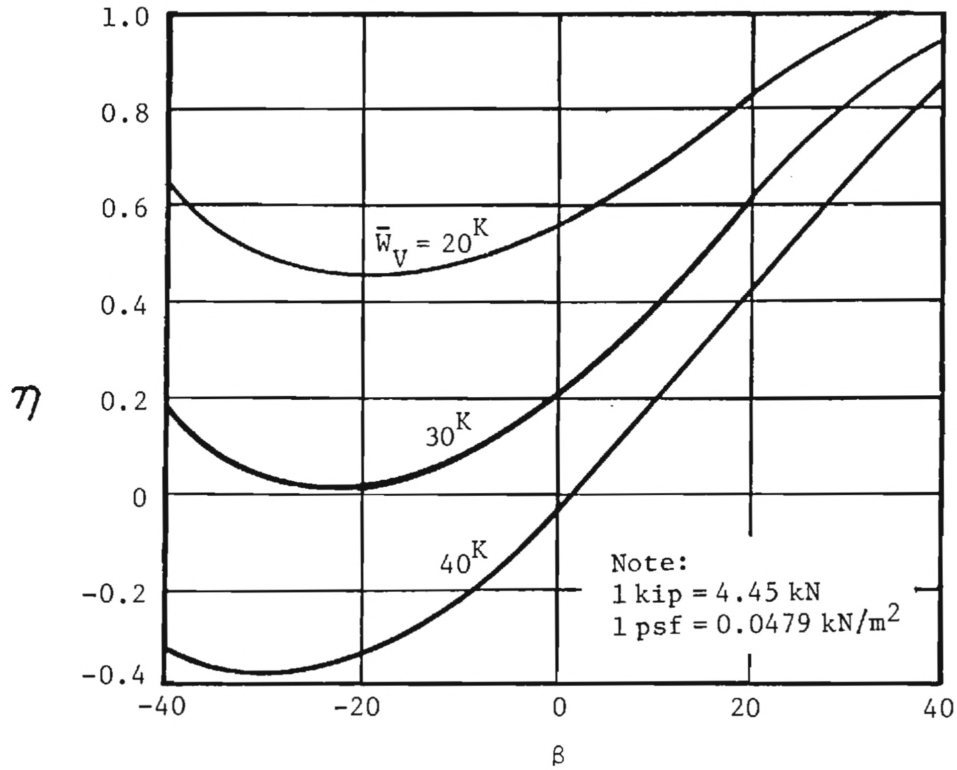

FIGURE 85. LOCAL BEARING FAILURE STABILITY REDUCTION FACTORS FOR DEEP FAILURE: $\phi = 30^o$, c = 100 PSF.

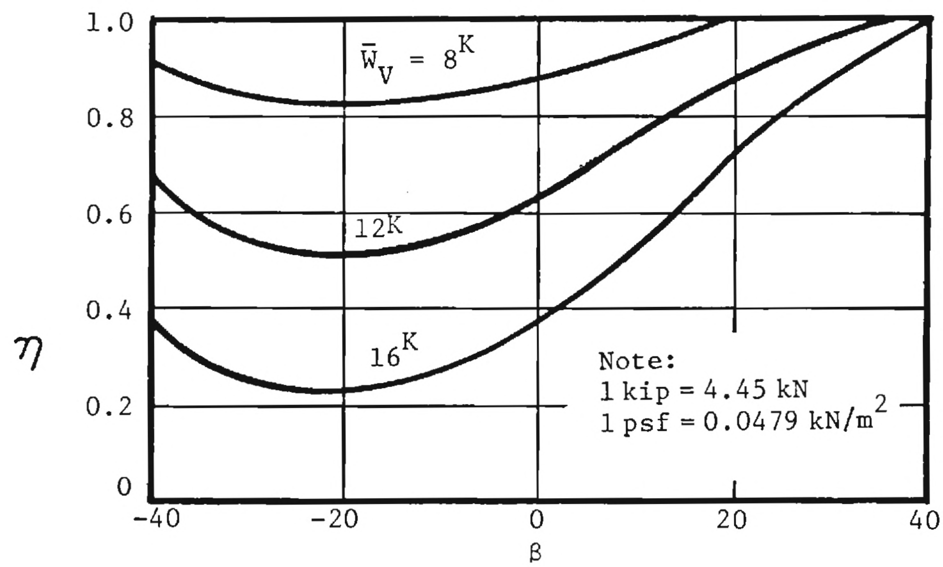

FIGURE 86. LOCAL BEARING FAILURE STABILITY REDUCTION FACTORS FOR SHALLOW FAILURE: $\phi = 30^o$, c = 100 PSF.

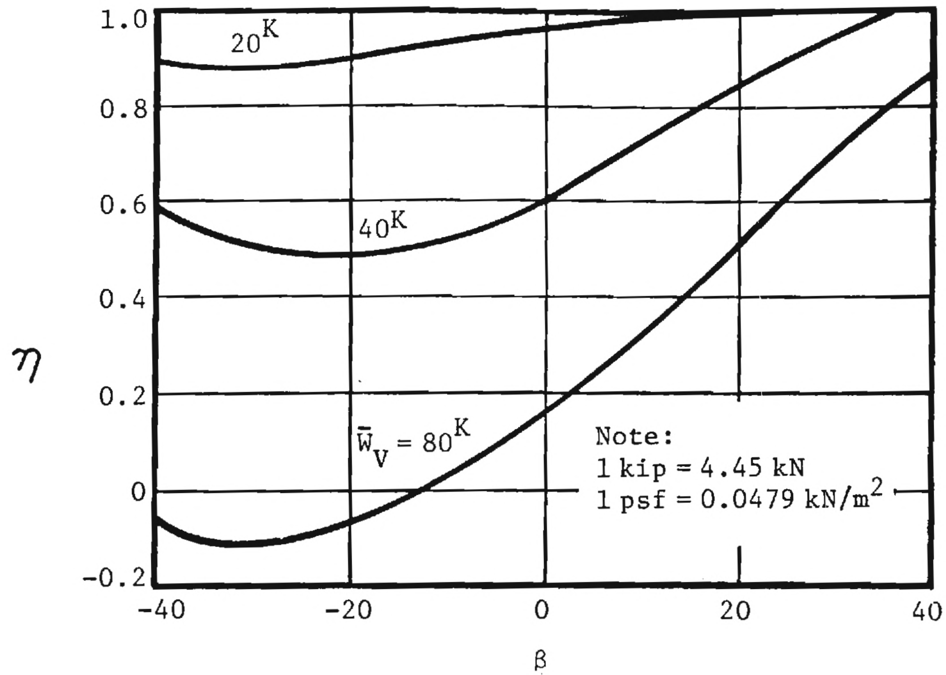

FIGURE 87. LOCAL BEARING FAILURE STABILITY REDUCTION FACTORS FOR DEEP FAILURE: $\phi = 36^o$, c = 200 PSF.

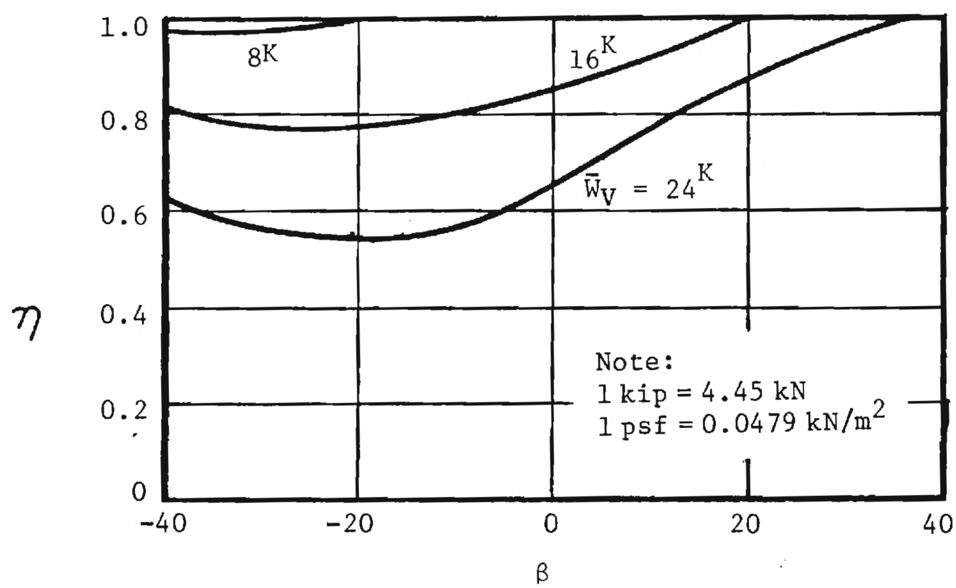

FIGURE 88. LOCAL BEARING FAILURE STABILITY REDUCTION FACTORS FOR SHALLOW FAILURE: $\phi = 36^o$, c = 200 PSF.

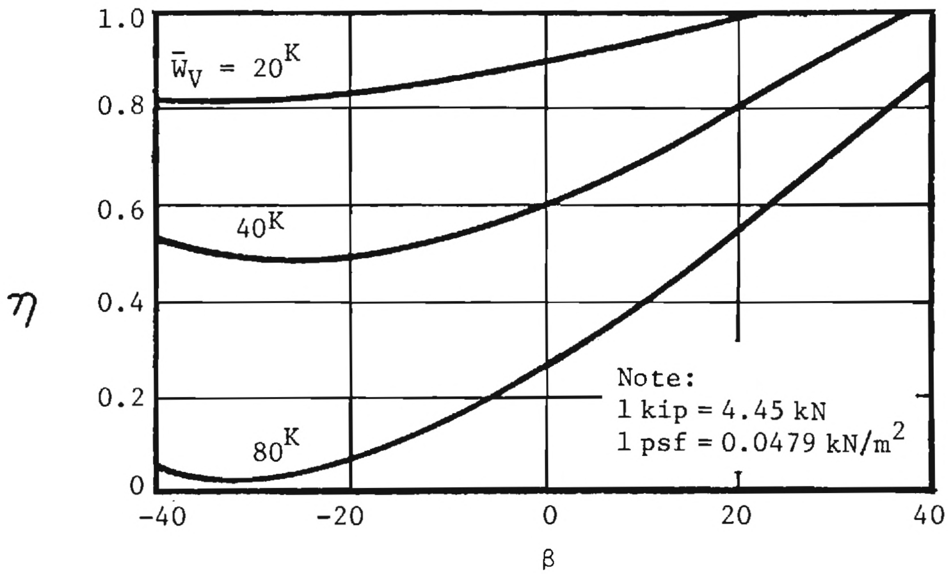

FIGURE 89. LOCAL BEARING FAILURE STABILITY REDUCTION FACTORS FOR DEEP FAILURE: $\phi = 42^o$, c = 200 PSF.

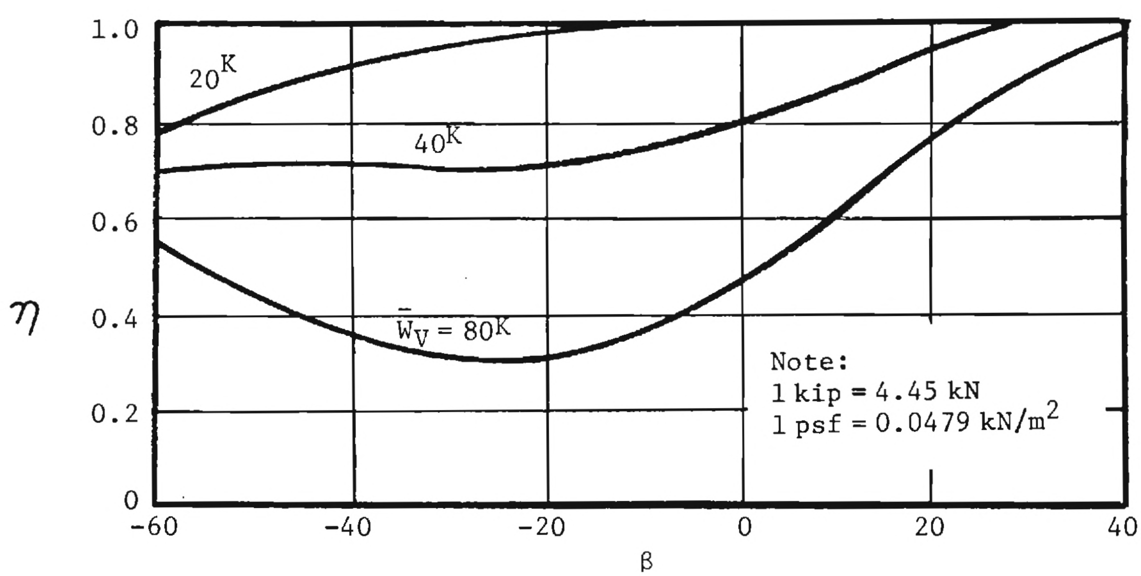

FIGURE 90. LOCAL BEARING FAILURE STABILITY REDUCTION FACTORS FOR DEEP FAILURE: $\phi = 42^o$, c = 300 PSF.

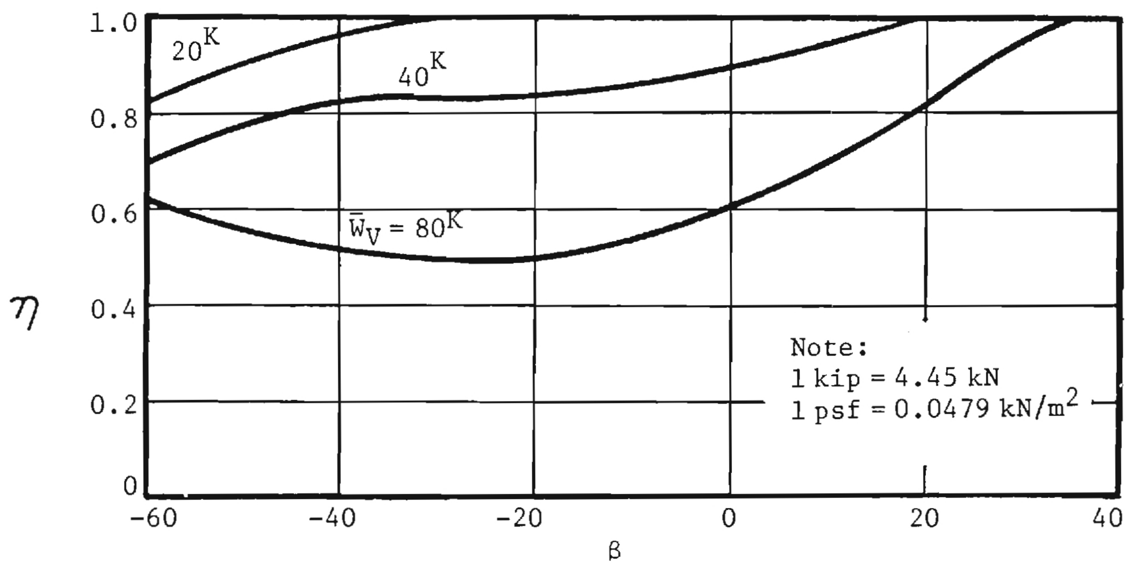

FIGURE 91. LOCAL BEARING FAILURE STABILITY REDUCTION FACTORS FOR DEEP FAILURE: $\phi = 42^{\circ}$, c = 400 PSF.

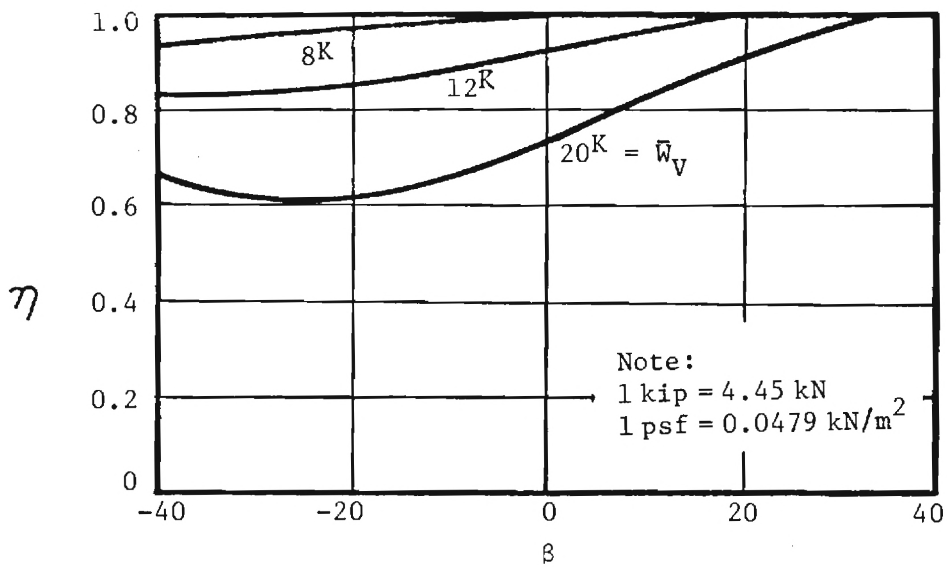

FIGURE 92. LOCAL BEARING FAILURE STABILITY REDUCTION FACTORS FOR SHALLOW FAILURE: $\phi = 42^o$, c = 200 PSF.

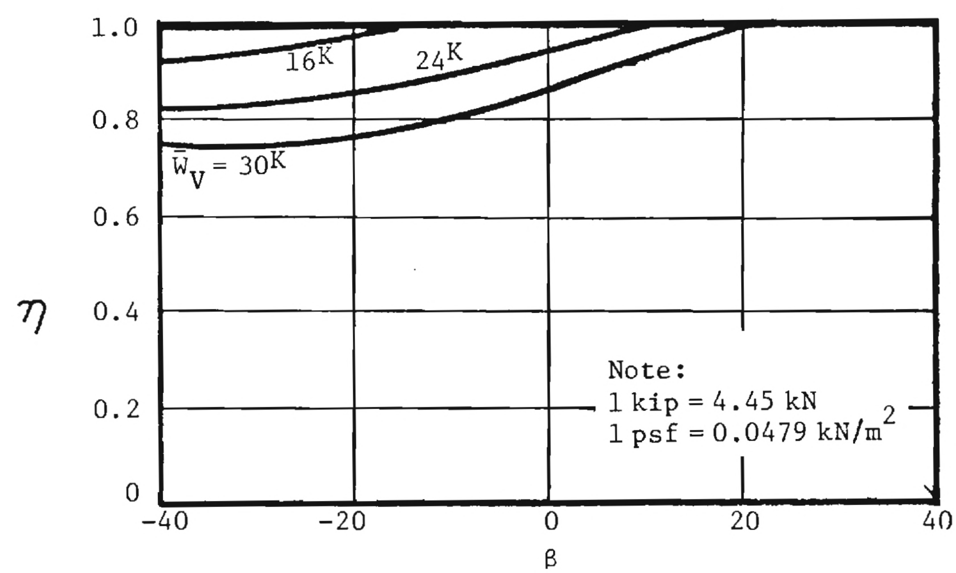

FIGURE 93. LOCAL BEARING FAILURE STABILITY REDUCTION FACTORS FOR SHALLOW FAILURE: $\phi = 42^o$, c = 400 PSF.

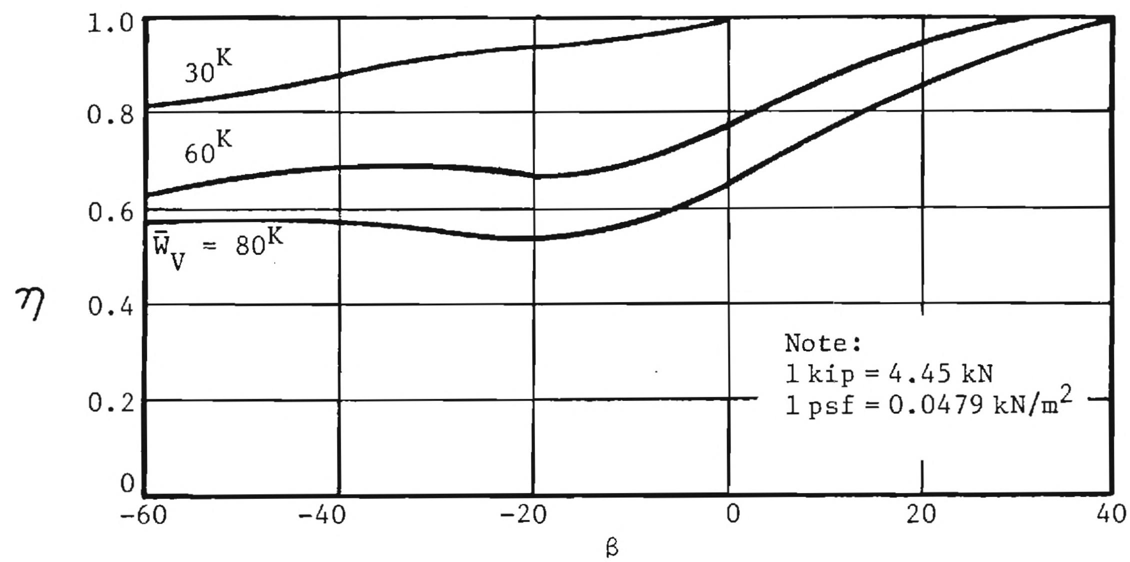

FIGURE 94. LOCAL BEARING FAILURE STABILITY REDUCTION FACTORS FOR DEEP FAILURE: $\phi = 45^o$, c = 600 PSF.

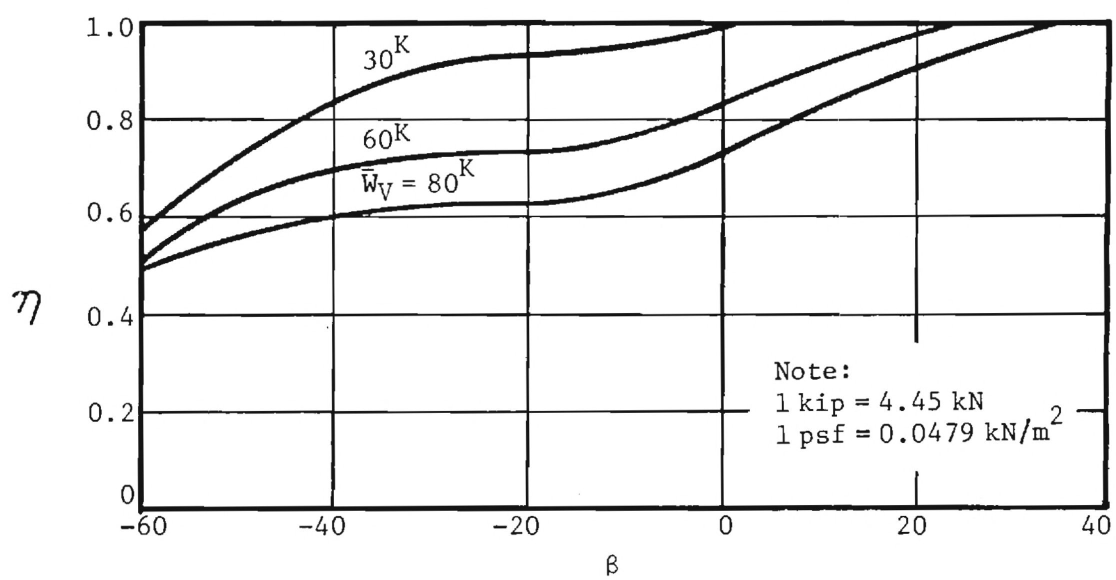

FIGURE 95. LOCAL BEARING FAILURE STABILITY REDUCTION FACTORS FOR DEEP FAILURE: $\phi = 50^o$, c = 800 PSF.

3. Using $\overline{W}_v$ from Step 2 and the design value(s) of ϕ_s and the cohesion of the clay c, enter the appropriate figure and estimate the value of the reduction factor η.

4. The "deep" charts should be used when the combined embankment height and stone column depth $h + z_o$ is equal to or greater than 3 stone column diameters; otherwise the "shallow" charts should be used.

The ratio η, obtained from these figures, is defined as follows:

$$\eta = \frac{F(\text{from equation 52})}{\overline{W}_N \tan\phi_s}$$

Physically η is the ratio of resisting force that is developed by an isolated stone column if a local bearing failure occurs to the force developed if local failure does not occur (i.e., the force that conventionally would be used in a stability analysis). Hence, η is the reduction factor indicating when a local bearing failure *may* become a problem for the given geometry and material properties used in the design. Theoretically, when $\eta < 1$ local bearing controls the maximum resisting force and moment that can be developed by the stone column. A reduction in resisting force (and moment) developed by the stone column would result in a reduction in safety factor of the slope compared to that computed for a general shear failure.

Design

Full-scale and model direct shear tests indicate a local bearing failure of at least a single stone column is possible. The analysis including the design curves just presented is for a single, isolated stone column. The relatively close proximity of adjacent stone columns and lateral spreading greatly complicate the actual problem compared with an isolated column; certainly further field and model tests are needed in addition to more refined theories. Nevertheless, the design charts and theory presented can be used to indicate when local bearing failure may be a problem. Further, the proposed approach is useful as a general guide in design for selecting safe design parameters (ϕ_s, n).

The likelihood of a local bearing failure increases as the shear strength of the clay decreases, and as a greater angle of internal friction ϕ_s and stress concentration factor n is used in design. For example, if an angle of internal friction, ϕ_s of the stone column of 42° is used, a local bearing *might* occur in cohesive soils having undrained shear strengths less than about 400 psf (19 kN/m^2) -- examine Figs. 89 through 93 for typical values of β and $\overline{W}_V$. A local bearing failure could occur in higher strength cohesive soils if ϕ_s values greater than 42° are used in design. Therefore, when stability is being analyzed in very soft and soft cohesive soils, the effect of a local bearing failure on the overall slope stability should be considered. Also, in firm and stiff soils such an analysis may show use of higher values of ϕ_s may be possible without undergoing a local bearing failure.

Local bearing failure can be easily handled in a slope stability analysis using the concept of a limiting angle of internal friction ϕ_s of the stone. Using this simplified approach several representative points are selected along the critical failure circle(s) as determined by a stability analysis on the stone column improved ground. The effective vertical stress, $\overline{W}_v$ and inclination of the failure circle β (with correct sign) at the selected points is determined. Figs. 85 through 95 can then be used to determine if a local bearing failure might occur at the selected points (and the actual magnitude of the reduction in the resisting shear force F). If a local failure is found not to occur over a significant portion of the failure surface, the design is satisfactory; otherwise consideration should be given to reducing ϕ_s. Note that the figures indicate local failure in general may be a problem only when $\beta<0$ (i.e., near and to the outside of the toe of the slope). Also, and perhaps more importantly, the charts serve to indicate when local failure is not of concern. In any case past experience and good engineering judgement should be taken into consideration in estimating the stability of the slope.

APPENDIX C

EXAMPLE BEARING CAPACITY PROBLEMS

Bearing Capacity Example 1

Example 1 illustrates prediction of the load due to a wide fill that can be supported by stone column improved ground to avoid a shear failure of the stone columns. The specific problem is to determine what height of fill the stone column improved ground can safely support. Both a general shear failure and a local bulging failure in a deep, very soft clay layer (Fig. 96) must be considered. The subsurface conditions and pertinent parameters needed to solve the problems are shown on Fig. 96. Assume the stone column has an angle of internal friction ϕ_s of 42^o, and an equilateral triangular pattern of columns is used having a spacing s = 7 ft. (2.1 m)

1. Calculate the area replacement ratio a_s from equation 5b:

$$a_s = 0.907 \left(\frac{D}{s}\right)^2 = 0.907 \left(\frac{3.5 \text{ ft.}}{7 \text{ ft.}}\right)^2 = 0.227 \qquad (5b)^*$$

$$A_s = \pi \cdot D^2/4 = 3.14(3.5 \text{ ft.})^2/4 = 9.62 \text{ ft.}^2$$

$$A = A_s/a_s = 9.62 \text{ ft.}^2/0.227 = 42.4 \text{ ft.}^2 \text{ (total area)} \qquad (3)^*$$

Note that all numbers in parentheses with an asterisk given to the side of an equation used in the example problems refer to equations given in the main text.

2. Stone Column. Estimate the general ultimate capacity of the stone column using equation (50) assuming a bulging failure occurs in the upper three stone column diameters of depth. Since the clay has a PI < 30 and is not classified as very soft (c < 250 psf), use $\tilde{N}_c$ = 22 (refer to Chapter VII).

$$\tilde{q}_{ult} = c\tilde{N}_c = 0.45 \text{ ksf } (22) = 9.9 \text{ ksf} \qquad (50)^*$$

$$P_{ult} = \tilde{q}_{ult} A_s = 9.9 \text{ ksf } (9.62 \text{ ft.}^2) = 95.2 \text{ k}$$

In the above expressions the stress in the stone column at ultimate is $\sigma_s = \tilde{q}_{ult} = c\tilde{N}_c$.

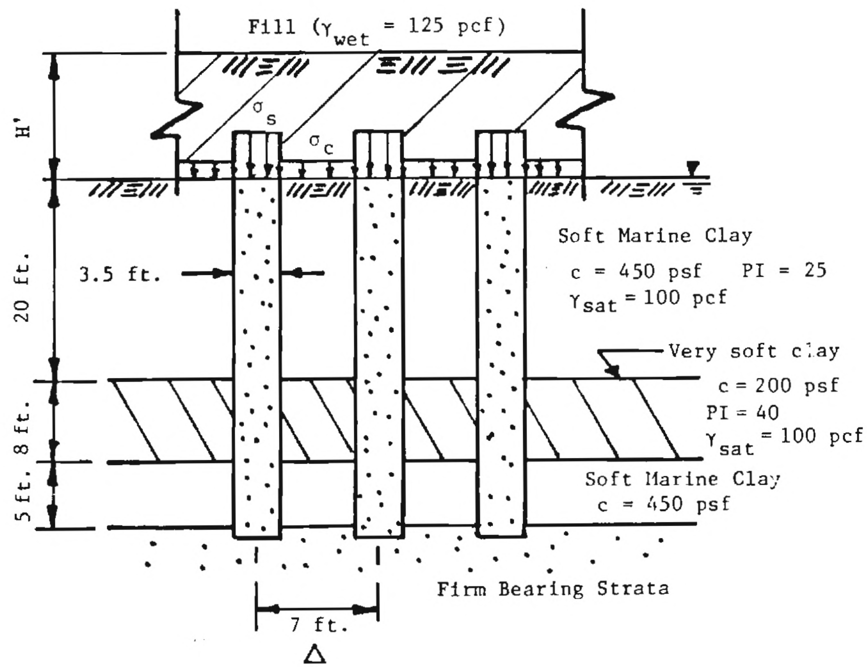

FIGURE 96. BEARING CAPACITY EXAMPLE 1 - WIDE FILL OVER STONE COLUMN IMPROVED CLAY.

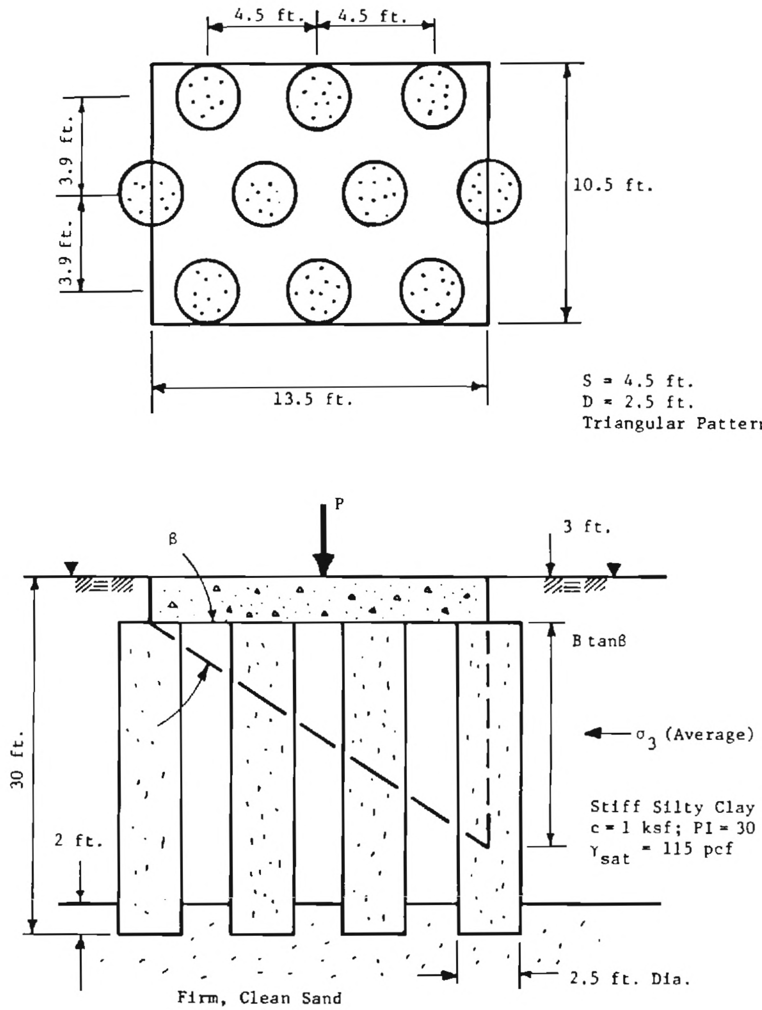

FIGURE 97. BEARING CAPACITY EXAMPLE 2.

3. Deep Bulging. Now check for the possibility of a bulging failure in the very soft clay stratum located at a depth of 20 ft. As discussed in Appendix B the ultimate lateral stress which an isolated stone column can develop is approximately equal to $\sigma_3 \cong 9c = 9(0.2 \text{ ksf}) = 1.8$ ksf since the weak stratum is greater than 3D below the surface. From equation (9) the ultimate stress the stone column can carry is then

$$\tilde{q}_{ult} = \sigma_3 (1 + \sin\phi_s)/(1 - \sin\phi_s) = 1.8 \text{ ksf } (5.04) \qquad (9)*$$

$$\tilde{q}_{ult} = 9.07 \text{ ksf}$$

Since the ultimate stress the stone column can carry considering a deep bulging failure in the very soft layer is slightly less than for a failure at the surface, the very soft deep stratum controls.

4. Cohesive Soil. The maximum ultimate stress the clay surrounding the stone column can take is $\sigma_c = 5c = 5(0.450 \text{ ksf}) = 2.25$ ksf. However, the total load applied to the unit cell must also not overload the clay. Assuming an $n = 3$, from equations (8a) and (8b)

$$\mu_s = n/[1 + (n-1)\, a_s] = 3/[1 + (3-1)\, 0.227] = 2.06$$

$$\mu_c = 1/[1 + (n-1)\, a_s] = 1/[1 + (3-1)\, 0.227] = 0.688$$

Then $\sigma_c \leq \mu_c \sigma = \mu_c (\sigma_s/\mu_s)$

$$\sigma_c < \mu_c \sigma = 0.688\,(9.07 \text{ ksf})/2.06 = 3.0 \text{ ksf}$$

Since 3.0 ksf is greater than $5c = 2.25$ ksf, $\sigma_c = 5c = 2.25$ ksf is the ultimate stress the clay can carry.

5. Allowable Fill Loading. The ultimate loading that can be applied over the unit cell area well within the fill area is

$$P_{ult} = \sigma_s A_s + \sigma_c A_c = (9.07 \text{ ksf})(9.62 \text{ ft.}^2) + (2.25 \text{ ksf})(32.8 \text{ ft.}^2)$$

$$P_{ult} = 161 \text{ k}$$

Using a safety factor of 2.0 the allowable loading is $P_{all} = 161 \text{ k}/2 = 80.5$ k. The height of embankment that will apply the safe loading to the unit cell is $\gamma_{wet}^{fill} H' = \sigma = P_{all}/A$.

Hence

$$H' = P_{all}/(\gamma_{wet}^{fill} \cdot A) = 80.5\,k/(0.125\,kcf \times 42.4\ ft.^2)$$

$$H' = 15.1\ ft.$$

6. Commentary. Settlement of the fill would be significant and should be calculated. Also, the stability at the edge of the fill should be checked using a circular arc analysis. In this example the very soft clay layer at a depth of 20 ft. controls the load that can be applied to the stone columns. Use of an ultimate lateral stress of 9c acting on the stone columns should give a conservative, but realistic, estimate of the ultimate resistance to bulging that can be developed (refer to Appendix B for a more indepth consideration of this aspect).

   Using $\sigma_3 = 9c$ as the limiting lateral pressure the soil can withstand, the ultimate load a stone column can carry would for $\phi_s = 42^o$ be equal for depths greater than 3D to $\tilde{q}_{ult} = 9c\,(1+\sin\phi_s)/(1-\sin\phi_s) = 9c\,(5.04) = 45c$ or $\tilde{N}_c = 45$ which indicates a limiting value of $\tilde{N}_c$ exists at a deep depth.

   Because the fill is wide, the stress on the stone column does not decrease with depth due to lateral spreading of stress. If a narrow group of stone columns had been used, the stress would, however, decrease with depth; this could be taken into account to determine the increased stress that could be applied at the surface compared with the level of the very soft clay stratum which controlled.

   Finally, Vesic cavity expansion theory could also have been used to determine the ultimate capacity of the stone column in the weak stratum. Since the clay is very soft and has a PI > 30, E = 5c is used to calculate a Rigidity Index, I_r (equation 13) of 1.72 for $\nu_s = 0.45$. In this analysis let q = the total lateral stress acting at the center of the soft layer. Nonlinear finite element analyses indicate the lateral pressure due to the applied surface loading σ_c can be conservatively approximated as $0.4\sigma_c$:

$$q = K_o \gamma z + 0.4\sigma_c = 0.75(24\ ft.)(0.1\ kcf) + 0.4\ (2.25\ ksf)$$

$$q = 2.7\ ksf$$

   Now $F'_c = \ell n\, I_r + 1 = \ell n\ 1.72 + 1 = 1.54$ for $\phi_c = 0$ and no volume change. Then the ultimate load the stone column can carry is

$$q_{ult} = [c\ F'_c + q\ F'_q](1+\sin\phi_s)/(1-\sin\phi_s) \qquad (14)*$$

$$q_{ult} = [0.2\,ksf\ (1.54) + 2.7\,ksf\ (1)][5.04] = 15.1\ ksf$$

Because of the large effect of overburden pressure, cavity expansion theory appears to overestimate the load which the stone column can carry through the very soft clay stratum.

Bearing Capacity Example 2: Square Group

Stone columns are to be used to improve a stiff clay to slightly reduce settlement of a foundation 13.5 ft. by 10.5 ft. (4.1 m x 3.2 m) in plan (Fig. 97). The modular ratio between the stone columns and the surrounding clay is estimated to be 6.0. Determine the ultimate and safe bearing capacity of the ten stone column group illustrated in Fig. 97. The material properties and geometries involved are shown on the figure.

From Fig. 27 the stress concentration in the stone column improved ground is about 2.0. The bearing capacity calculations are as follows:

1. Calculate the area replacement ratio, a_s

$$A_s = \frac{3.14}{4} (2.5 \text{ ft.})^2 \times 10 = 49.1 \text{ ft.}^2; \quad A = 13.5 \text{ ft.} \times 10.5 \text{ ft.} = 141.8 \text{ ft.}^2;$$

$$a_s = A_s/A = 49.1 \text{ ft.}^2/141.8 \text{ ft.}^2 = 0.346$$

2. Determine the stress concentration in the stone column from equation (8b) (or Fig. 68):

$$\mu_s = n/[1+(n-1)\,a_s] = 2/[1+(2-1)(0.346)] = 1.49 \qquad (8b)*$$

3. Calculate the composite shear strength within the stone column group (equation 16a and equation 16b) and related parameters.

$$[\tan\phi]_{avg} = \mu_s \tan\phi_s \,(a_s) = (1.49) \tan 42.0^o (0.346) = 0.464 \qquad (16a)*$$

$$\phi_{avg} = \tan^{-1} (0.464) = 24.9^o$$

$$\tan\beta = 1.566 \quad \tan^2\beta = 2.454$$

$$c_{avg} = c \times (1-a_s) = 1 \text{ ksf } (1-0.346) = 0.654 \text{ ksf} \qquad (16b)*$$

4. Using Vesic cavity expansion theory, calculate the ultimate lateral stress σ_3 in the clay surrounding the stone column group. Since the clay is stiff, has no organics and has a PI = 30, use E = 11c for calculating the Rigidity Index, I_r. The average diameter of the foundation is $B = \sqrt{4A/3.14} = 13.4$ ft. The depth of the failure wedge is then (Fig. 97) $B \tan\beta + 3$ ft. = (13.4 ft.)(1.566) + 3 ft. = 24 ft. The initial lateral stress in the stiff silty clay surrounding the stone columns will be used as a conservative estimate of the mean stress q (equation 12),

for use in the cavity expansion theory. The stiff silty clay is known to be normally consolidated. Therefore from reference 62, p.300, $K_o \cong 0.6$ for the surrounding silty clay, and $q \doteq 0.6$ (13.5 ft. x 0.115 kcf) = 0.931 ksf. Now calculate the Rigidity Index (equation 13):

$$I_r = \frac{E}{2(1+\nu)(c+q\ \tan\phi)} = \frac{11c}{2(1+0.45)(c+q\ \tan 0^o)} \qquad (13)^*$$

giving $I_r = 3.79$. From $F'_c = \ell n\, I_r + 1$ for $\phi = 0$ and Fig. 19, $F'_c = 2.33$ and $F'_q = 1.0$. Then calculate the ultimate lateral stress which can be developed by the surrounding silty clay:

$$\sigma_3 = c\ F'_c + q\ F'_c = 1\ \text{ksf}\ (2.33) + 0.931\ \text{ksf}\ (1.0) = 3.26\ \text{ksf} \qquad (12)^*$$

5. Calculate the ultimate vertical stress and load that can be applied over the rigid foundation (see equation 19 in text):

$$q_{ult} = \sigma_1 = \sigma_3\ \tan^2\beta + 2c_{avg}\ \tan\beta = 3.26\ \text{ksf}\ (2.454) + 2(0.654\ \text{ksf})$$

$$(1.566) \qquad (19)^*$$

$$q_{ult} = 8.0\ \text{ksf} + 2.0\ \text{ksf} = 10.0\ \text{ksf}$$

The ultimate load that can be carried by the foundation is $P_{ult} = q_{ult} \cdot A = 10.0$ ksf (141.8 ft.^{2}) = 1418 k. Using a safety factor of 2.0, the foundation can carry P_{ult} = 1418 k/2.0 = 709 k. This amounts to 70.9 k (or 35.5 tons) per stone column if the silty clay is assumed not to carry any of the load. This level of loading is reasonable for a foundation where settlement is of concern (refer to Table 12).

6. Commentary. Settlement of course would control the design. A total load on the group of 709 k would be used for a first settlement estimate. For this loading, the average stress applied to the foundation is σ = 709 k/141.8 ft.^{2} = 5 ksf. The probable distribution of stress between the stone and soil for n = 2 would be

$$\sigma_c = \mu_c \sigma = 0.743\ (5\ \text{ksf}) = 3.7\ \text{ksf}$$

$$\sigma_s = \mu_s \sigma = 1.49\ (5\ \text{ksf}) = 7.45\ \text{ksf}$$

Since the ultimate stress of the stiff clay is about 6.2c = 6.2 ksf, the stress level in the clay is not excessive. Using the proposed design, the ratio of the settlement of the treated to unimproved ground would approximately be $S_t/S \cong \mu_c = 0.74$ (refer to equations 20, 21 and 22). Thus for the conditions analyzed, reduction in settlement on the order of 25 percent would be expected. For the given site conditions, use of a larger footing (without stone columns) should also be evaluated considering magnitude of settlements and the economics of the designs.

In general for the wet method a stone column spacing less than 5 ft. is not recommended; Example Problem 2 would therefore be an exception because of the presence of the stiff, silty clay.

APPENDIX D

EXAMPLE SETTLEMENT PROBLEMS

Settlement Example 1

Settlement Example 1 illustrates calculating settlements of a soft clay reinforced with stone columns and loaded by a wide fill. The calculation of the load carrying capacity of stone column improved ground for a problem similar to this was illustrated by Example 1 in Appendix C. In the present example, primary consolidation settlements are calculated using both the equilibrium and finite element methods. Secondary settlements are also calculated. The problem is illustrated in Fig. 98. The site consists of 20 ft. (6.1 m) of gray, soft silty clay overlying a firm to dense sand. The groundwater table is at the surface. An equilateral triangular pattern of stone columns is used having a spacing of 6.5 ft. (2 m). The diameter of the stone column is estimated (Table 13) to be 3.5 ft. (1.07 m). A 2.5 ft. (0.7 m) sand blanket is to be placed over the soft silty clay for a working platform and drainage blanket.

Equilibrium Method. The average stress σ exerted by the 2.5 ft. sand blanket and 12.5 ft. structural fill on the top of the stone columns is σ = 12.5 ft. x 120 pcf + 2.5 ft. (108 pcf) = 1770 psf. The area replacement ratio, a_s from equation (5b) is for an equilateral, triangular stone column pattern

$$a_s = 0.907\,(D/s)^2 = 0.907\left(\frac{3.5\text{ ft.}}{6.5\text{ ft.}}\right)^2 = 0.263 \qquad (56)*$$

Assume for the *settlement analysis* the stress concentration factor n to be 5.0. Then the stress concentration factor μ_c in the clay is from equation (8a) or Fig. 68

$$\mu_c = 1/[1+(n-1)\,a_s] = 0.487 \qquad (8a)*$$

The initial effective stress $\bar{\sigma}_o$ at the center of the silty clay layer is

$$\bar{\sigma}_o = 10\text{ ft.} \times (95\text{ pcf} - 62.4\text{ pcf}) = 326\text{ psf}$$

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* These numbers refer to equations previously given in the main text.

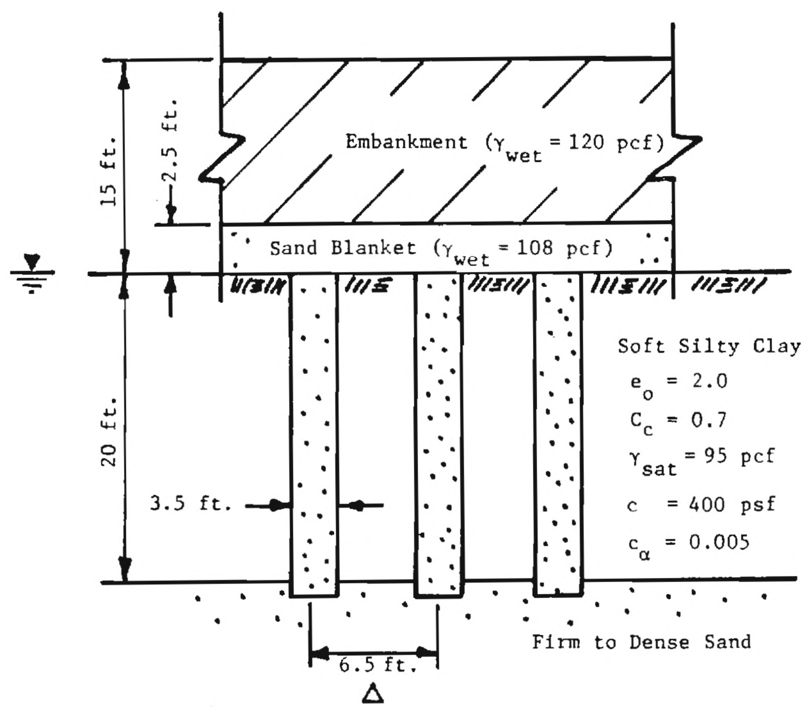

FIGURE 98. SETTLEMENT EXAMPLE 1 - WIDE FILL OVER STONE COLUMN IMPROVED SILTY CLAY.

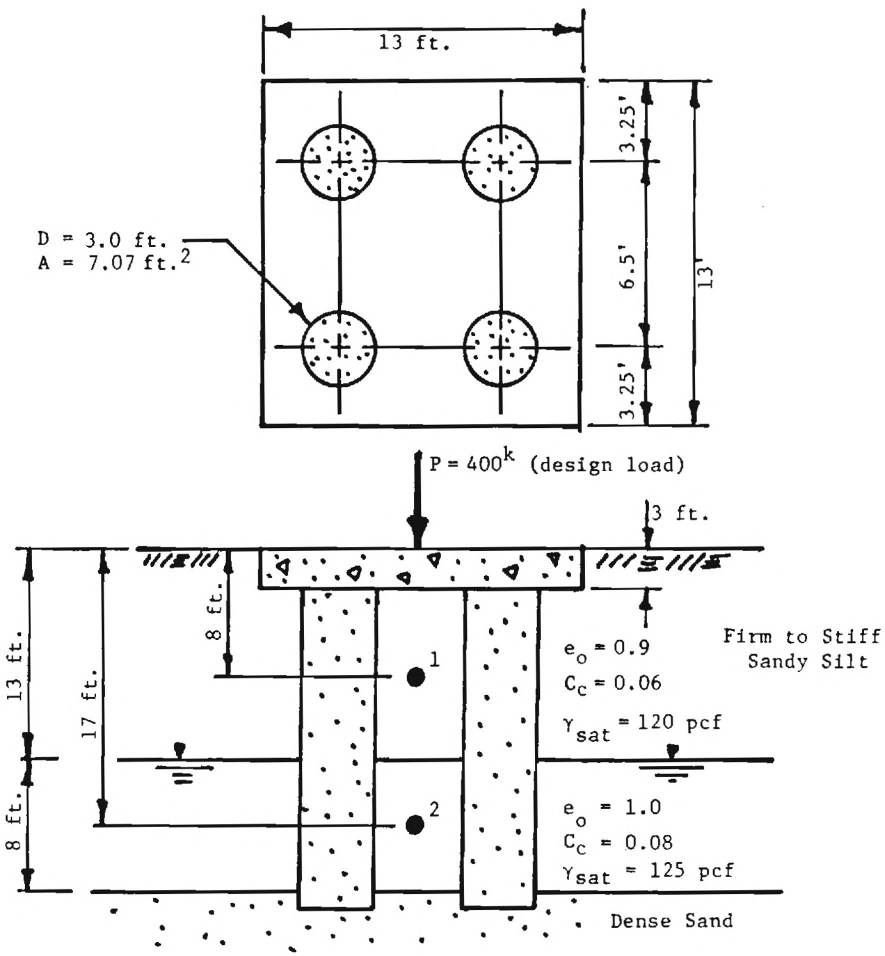

FIGURE 99. SETTLEMENT EXAMPLE 2 - RIGID FOUNDATION PLACED OVER STONE COLUMN IMPROVED SANDY SILT.

The primary consolidation settlement in the clay layer from one-dimensional consolidation theory is from equation (20)

$$S_t = \frac{C_c}{1+e_o} \log_{10} \left(\frac{\bar{\sigma}_o + \sigma_c}{\bar{\sigma}_o}\right) \cdot H \qquad (20)*$$

$$S_t = \left(\frac{0.7}{1+2.0}\right) \log_{10} \left(\frac{326 \text{ psf} + (1770 \text{ psf})(0.487)}{326 \text{ psf}}\right) \cdot (20 \text{ ft.} \times 12 \text{ in./ft.})$$

$$S_t = 31.4 \text{ in.}$$

The estimated primary consolidation settlement of the stone column improved silty clay layer is thus 31 in. following the equilibrium method. For comparison, the settlement in the silty clay layer if not improved with stone columns would be 45.2 in.

Note how simple the equilibrium method is to apply to a problem. The "trick", of cource, is to estimate the correct value of stress concentration factor n to use in the analysis. In this problem the fill was wide and no dissipation laterally of stress with depth occurs. The next settlement example shows how both the equilibrium and the finite element methods can be applied to a problem where the applied stress decreases with depth.

Nonlinear Finite Element Method. Since the clay is soft and quite compressible use the nonlinear finite element method of analysis. First calculate the modulus of elasticity E_c of the clay for the approximate stress range of interest. The initial average stress in the clay from the equilibrium method is $\bar{\sigma}_o = 326$ psf. The change in stress in the clay due to the embankment loading is $\sigma_c = \mu_c \sigma$ = 0.559 (1770) psi) = 989 psf. Using Table 10 and experience as a guide, the drained Poisson's ratio of the clay is assumed to be 0.42 from equation (47). The modulus of elasticity of the clay for the applicable stress range is

$$E_c = \frac{(1+\nu)(1-2\nu)(1+e_o)\,\sigma_{va}}{0.435\,(1-\nu)\,C_c} = \frac{(1+0.42)(1-2 \times 0.42)(1+2.0)}{0.435\,(1-0.42)(0.70)} \; \frac{(326 \text{ psf} + 989 \text{ psf})}{2}$$

$$E_c = 2538 \text{ psf} = 17.6 \text{ psi}$$

Note that the value of Poisson's ratio selected has a significant effect on the calculated value of E_c: larger values of ν_c give smaller values of E_c.

The stone column length to diameter ratio in the soft clay is, L/D = 20 ft./ 3.5 ft. = 5.7. The average applied pressure σ due to the embankment is σ = 1770 psf = 12.3 psi. Interpolating from Figs. 32 and 33 ($a_s = 0.25$) for a soft boundary condition ($E_b = 12$ psi), the effective ratio of settlement of stone column reinforced ground to stone column length, S/L = 0.078. (In interpolating between figures for different S/L values, work in terms of settlement since the length varies). The embankment settlement from the finite element method is then S = 0.078 (20 ft. x 12 in./ft.) = 19 in. The "best" settlement estimate is the average of the finite element and incremental methods

$$S_t = (19 \text{ in.} + 31 \text{ in.})/2 = 25 \text{ in.}$$

The estimated reduction in settlement due to stone column improvement is then S_t/S = 25.0 in./45.2 = 0.586.

Time Rate of Settlement. Determine the magnitude of primary consolidation settlement after 2 months assuming instantaneous construction[1]. The silty clay has a vertical coefficient of consolidation C_ν of 0.05 ft.^{2}/day. Based on a detailed study of the strata, the horizontal permeability is estimated to be 3 times the vertical permeability. Then from equation (49)

$$C_{\nu_r} = C_\nu (k_h/k_v) = 0.05 \text{ ft.}^2/\text{day}\ (3) = 0.15 \text{ ft.}^2/\text{day} \qquad (49)^*$$

Assume the reduced drain diameter D' to account for smear is 1/5 the constructed stone column diameter. For an equilateral triangular stone column spacing s of 6.5 ft., the equivalent diameter D_e of the unit cell is

$$D_e = 1.05s = 1.05\,(6.5 \text{ ft.}) = 6.83 \text{ ft.} \qquad (1)^*$$

and

$$n^* = r_e/r_w = D_e/D' = 6.83 \text{ ft.}/(3.5 \text{ ft.}/5) = 9.76 \quad (\text{Fig. } 45)$$

The dimensionless vertical and horizontal time factors are then

$$T_z = C_\nu t/(H/N)^2 = 0.05 \text{ ft.}^2/\text{day}\ (2 \times 31 \text{ days})/(20 \text{ ft.}/2)^2 = 0.031 \qquad (27)^*$$

$$T_r = C_{\nu_r} t/(D_e)^2 = 0.15 \text{ ft.}^2/\text{day}\ (2 \times 31 \text{ days})/(6.83 \text{ ft.})^2 = 0.199 \qquad (28)^*$$

From Fig. 42, U_z = 0.12 and from Fig. 43, U_r = 0.64. The combined degree of consolidation, equation (25), is

1. Methods for handling construction over a finite time interval are given elsewhere [88].

$$U = 1-(1-U_z)(1-U_r) = 1-(1-0.12)(1-0.64) = 0.68 \qquad (25)^*$$

An important portion of the total primary consolidation settlement occurs after 2 months and equals $S_t' = 25.0$ in.$(0.68) = 17$ in. For this example problem having stone columns, vertical drainage had little effect on the time rate of primary settlement, due to the higher radial coefficient of consolidation and smaller radial drainage path to the vertical drains. For comparison, if stone columns had not been used, primary consolidation settlement would have been only 12 percent complete, with the primary settlement at the end of two months being only about 3 in.

Secondary Compression Settlement. Estimate the magnitude of secondary compression settlement that would be expected to occur 5 years after construction. Assume secondary compression begins at the time for 90 percent primary consolidation. Neglect the effects of vertical drainage which were shown above to be small. The radial time factor for 90 percent primary consolidation for $n^* = 9.76$ is $U_r = 0.47$ from Fig. 43. From Equation (28) the time for 90 percent primary consolidation is $t = T_r(D_e)^2/C_{v_r} = 0.47$ $(6.83 \text{ ft.})^2/(0.15 \text{ ft.}^2/\text{day}) = 146$ days after construction. The secondary compression settlement is then

$$\Delta S = C_\alpha H \log_{10}(t_2/t_1) \qquad (30)^*$$

$$\Delta S = 0.005(240 \text{ in.}) \log_{10}(5(365 \text{ days})/146 \text{ days}) = 1.3 \text{ in.}$$

For the silty clay in this problem, the secondary settlement is thus relatively small compared to a primary consolidation settlement of 26.5 in. If organics had been present secondary settlement would have been significantly greater.

Settlement Example 2

Settlement Example 2 illustrates how to handle, at least approximately, stress distribution in calculating settlement of stone column improved ground. Stone column improved ground is being considered as one design alternative for a slightly marginal site consisting of firm to stiff sandy silt as shown in Fig. 99. The average contact stress is $\sigma = P/A = 400$ kips (13 ft. x 13 ft.) = 2367 psf. The gross area replacement ratio from equation (3) is $a_s = A_s/A$ = (7.07 ft^2)(4)/(13 ft. x 13 ft.) = 0.167. Now determine the initial effective stress at the center of each layer:

Layer 1: $\bar{\sigma}_o$ = 8 ft. (120 pcf) = 960 psf

Layer 2: $\bar{\sigma}_o$ = 13 ft. (120 pcf) + 4 ft. (125 pcf - 62.4 pcf)

= 1810 psf

Calculate the change in stress $\Delta\sigma_z$ at the center of each layer using as an approximation Boussinesq stress distribution theory for a square foundation and the average applied stress, σ[1]:

Layer 1: z/B = 5 ft./13 ft. = 0.38B; $\Delta\sigma_z = I_z \cdot \sigma$ = 0.82 (2367 psf)

$\Delta\sigma_z$ = 1941 psf

Layer 2: z/B = 14 ft./13 ft. = 1.08B; $\Delta\sigma_z = I_z \cdot \sigma$ = 0.31(2367 psf)

$\Delta\sigma_z$ = 734 psf

The change in stress $\Delta\sigma_z$ calculated above is the average stress change over the unit cell.

Assume a stress concentration factor $n = 3$ (an n value less than 4 is used because the soil is relatively stiff compared with soft clays). The stress in the sandy silt from equation (8a) is

$$\mu_c = 1/[1 + (n-1)\, a_s] = 1/[1 + (2.0)(0.167)] = 0.750$$

The stress change in the clay as an approximation can be taken to equal $\mu_c \Delta\sigma_z$ giving the following settlements for Layers 1 and 2:

$$S_1 = \frac{0.06}{1+0.9} \log_{10}\left(\frac{960 \text{ psf} + 1941 \text{ psf } (0.750)}{960 \text{ psf}}\right) (10 \text{ ft.} \times 12 \text{ in.}) = 1.5 \text{ in.}$$

---

1. For a discussion of stress distribution and charts, tables, etc. for calculating changes in stress due to foundation loadings, refer to standard textbooks on soil mechanics [c.f., 62, 65, 74, 88].

$$S_2 = \frac{0.08}{1+1.0} \log_{10} \left(\frac{1810 \text{ psf} + 734 \text{ psf } (0.750)}{1810 \text{ psf}} \right) (8 \text{ ft.} \times 12 \text{ in.}) = 0.44$$

The total settlement in the sandy silt strata is about $S_t = 1.9$ in. Had stone columns not been used, the settlement would have been $S = 2.4$ in., giving $S_t/S = 1.9/2.4 = 0.79$. From equation (22), $S_t/S \geq \mu_c = 0.75$, which illustrates that $S_t/S \cong \mu_c$ is a quite useful approach for preliminary estimates of the level of reduction of settlement for various stone column designs. In the above simplified equation the variables affecting the settlement ratio S_t/S are only a_s and n.

Stress distribution can also be approximately considered using the finite element design charts. To do this an average stress σ is calculated within the compressible layer and used in the chart rather than the stress actually applied at the top of the layer.

Time Rate of Primary Consolidation. In Settlement Example 2, only four stone columns are used. Also, two layers of sandy silt are present which would have different coefficients of consolidation. Assume c_v (and c_{v_r}) in one layer differs from c_v (and c_{v_r}) in the other layer by a factor of about 2 to 5. For the resulting complex three-dimensional flow conditions, a theoretically accurate evaluation of the time rate of settlement for this problem would be a major undertaking. Such a solution would require a three-dimensional numerical analysis. As a rough, engineering approximation, however, the following simplified approach can be taken:

1. Consider for each layer radial and vertical flow separately and use equation (25) to estimate the combined results.

2. For radial flow neglect any interaction between the two layers. Sketch in the approximate radial flow paths on a scale drawing (Fig. 100). Remember that flow originates from lines of geometric symmetry and moves approximately radially to the drains.

   Consider the flow to the drain shown in the upper lefthand corner of Fig. 100. An examination of the flow paths on the figure show 25 percent of the flow to the stone column from quadrant a-o-b is from infinity. This means D_e for this quadrant is very large, and from equation (28) the radial time factor $T_r = 0$. Over quadrants b-o-c and a-o-e, which together comprises another 25 percent of the drain, the flow path length varies from infinity at points b and a to short drainage paths at points c and e; this combined quadrant will only be partially effective in providing drainage. Finally, the area contributing flow to the drain that lies to the right and below line c-o-e has short flow paths that can be approximated by an estimated equivalent unit cell diameter $D_e \cong 7.5$ ft. shown in dashed lines on the figure. As an engineering approximation for this example,

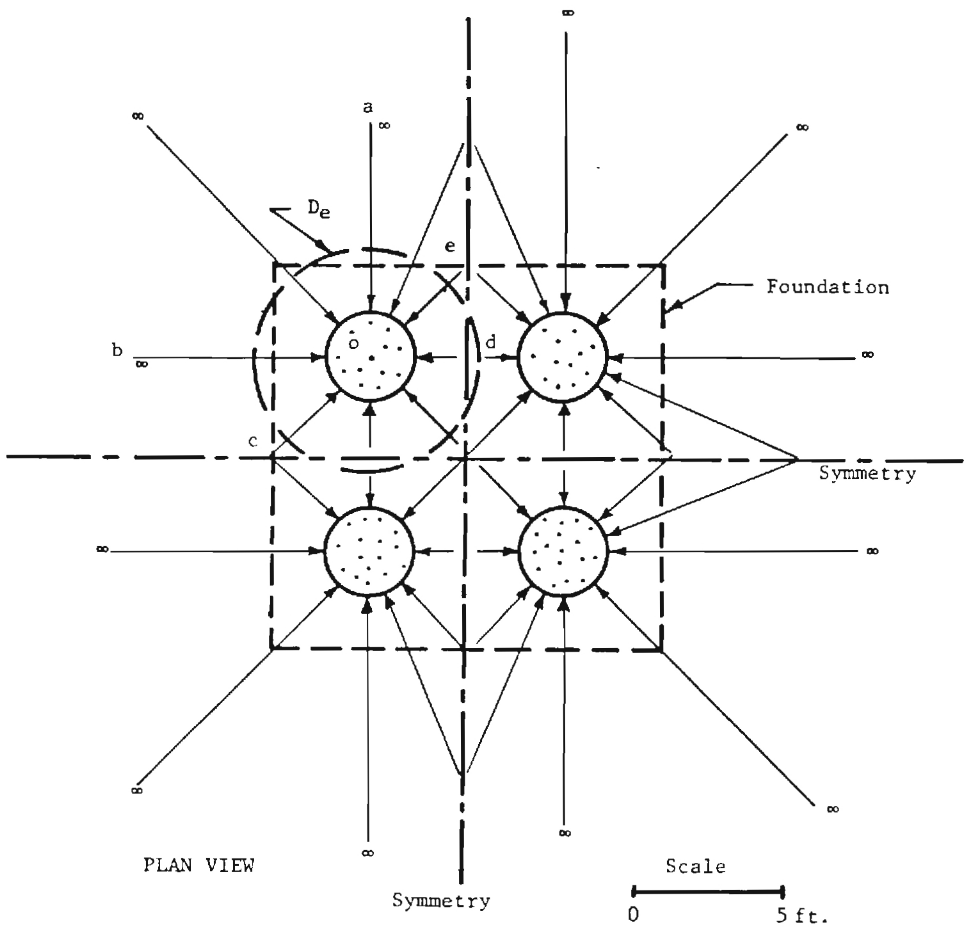

FIGURE 100. APPROXIMATE RADIAL FLOW PATHS FOR SETTLEMENT EXAMPLE 2.

estimate the time factor T_r for each layer using the appropriate value of c_{v_r} and $D_e \cong 7.5$ ft. To crudely consider that D_e effectively is very large over between 25 to 50 percent of the drain, reduce the time factor by about $(25\% + 50\%)/2 = 37.5\%$ or 0.4 (multiply the calculated time factor T_r by 0.6 or use 0.5 to be a little more conservative).

To illustrate this approximate approach assume for Layer 1 $c_{v_r} = 5c_v$ and $c_v = 0.2$ ft.^{2}/day. Then at the end of 2 months the radial time factor would be estimated from equation (28) as $T_r = c_{v_r} \cdot t/D_e^2 = 5(0.2 \text{ ft.}^2/\text{day})(2 \times 31 \text{ days})/(7.5 \text{ ft.})^2 = 1.10$. Reducing the time factor to approximately consider partial drainage gives $T_r = 0.5\ (1.10) = 0.551$. Assume the stone column diameter is effectively reduced by 1/5 to account for smear, giving from equation (29a) $n^*_{equiv} = 7.5 \text{ ft.}/(3 \text{ ft.} \times 0.2) = 12.5$. Then from Fig. 43 the degree of radial consolidation $U_r = 0.91$. Conservatively neglecting vertical drainage in Layer 1, the settlement after 2 months of Layer 1 is $S \cong 1.5$ in. $(0.9) = 1.35$ in. As would be expected, consolidation occurs rapidly in the sandy silt.

3. Since in this example $c_{v_r} = 5\ c_v$ for Layer 1, vertical compared to radial consolidation would be relatively slow, and was conservatively neglected. However, if the effect of vertical drainage on the time rate of consolidation is desired, the presence of two layers greatly complicates vertical time rate of consolidation computations. If c_v of the more permeable layer is more than 20 times c_v of the less permeable layer, the following simplified approach can be used [62, p.415]: (1) Assume consolidation occurs in two stages, (2) In Stage 1, calculate consolidation of the more permeable layer, assuming no drainage at the interface between the two, (3) In Stage 2 calculate consolidation in the least permeable layer, assuming drainage at the interface. If c_v of one layer is less than 20 times c_v of the other, the approximate method described in NAVFAC DM-7 [86] can be followed or numerical methods can be used [62, p.415].

4. <u>Commentary</u>. The above methods are, of course, quite crude and should be considered "ball park" in accuracy. They do give a rational way of approaching a very complicated, three-dimensional time rate of primary consolidation problem.

APPENDIX E

EXAMPLE STABILITY PROBLEM

This example illustrates how to handle the geometric and material parameters required for setting up a slope stability problem for analysis using the Profile Method described in Chapter III.

A 15 ft. (4.6 m) high embankment is to be placed over a soft clay as illustrated in Fig. 101. Because of the low shear strength of the soft clay use a stress concentration factor n of 2.0, and an angle of internal friction ϕ_s of the stone column of 42^o. The saturated unit weight of the stone is 125 pcf (19.6 kN/m^3). For the first trial design, use 5 rows of stone columns laid out as shown in Fig. 101. An equilateral triangular grid will be used having a trial spacing $s = 6.5$ ft. (2 m). The stone column diameter is estimated to be 3.5 ft. (1.07 m) giving an area replacement ratio of

$$a_s = 0.907\,(3.5/6.5)^2 = 0.263 \qquad (5b)*$$

The plan view of the stone column grid used to improve the site is shown in Fig. 101(b). As shown in the figure, stone columns replace only 26 percent of the total volume of the soft clay (i.e., $a_s = 0.263$). Further, in performing a conventional stability analysis, the materials are assumed to extend for an infinite distance in the direction of the embankment. Typically the analysis is then performed on a 1 ft. (0.3 m) wide slice of embankment. To use the profile method the discrete stone columns must therefore be converted into equivalent stone column strips extending along the full length of the embankment as follows:

$$A_s = \pi D^2/4 = 3.14\,(3.5\text{ ft.})^2/4 = 9.62\text{ ft.}^2$$

The length tributary to each stone column in the direction of the embankment is s = 6.5 ft. (2 m). Therefore a solid strip having the same area and volume of stone would have a width w of

$$w = A_s/s = 9.62\text{ ft.}^2/6.5\text{ ft.} = 1.48\text{ ft.}$$

The total width of the tributary area equals

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* These numbers refer to equations previously given in the main text.

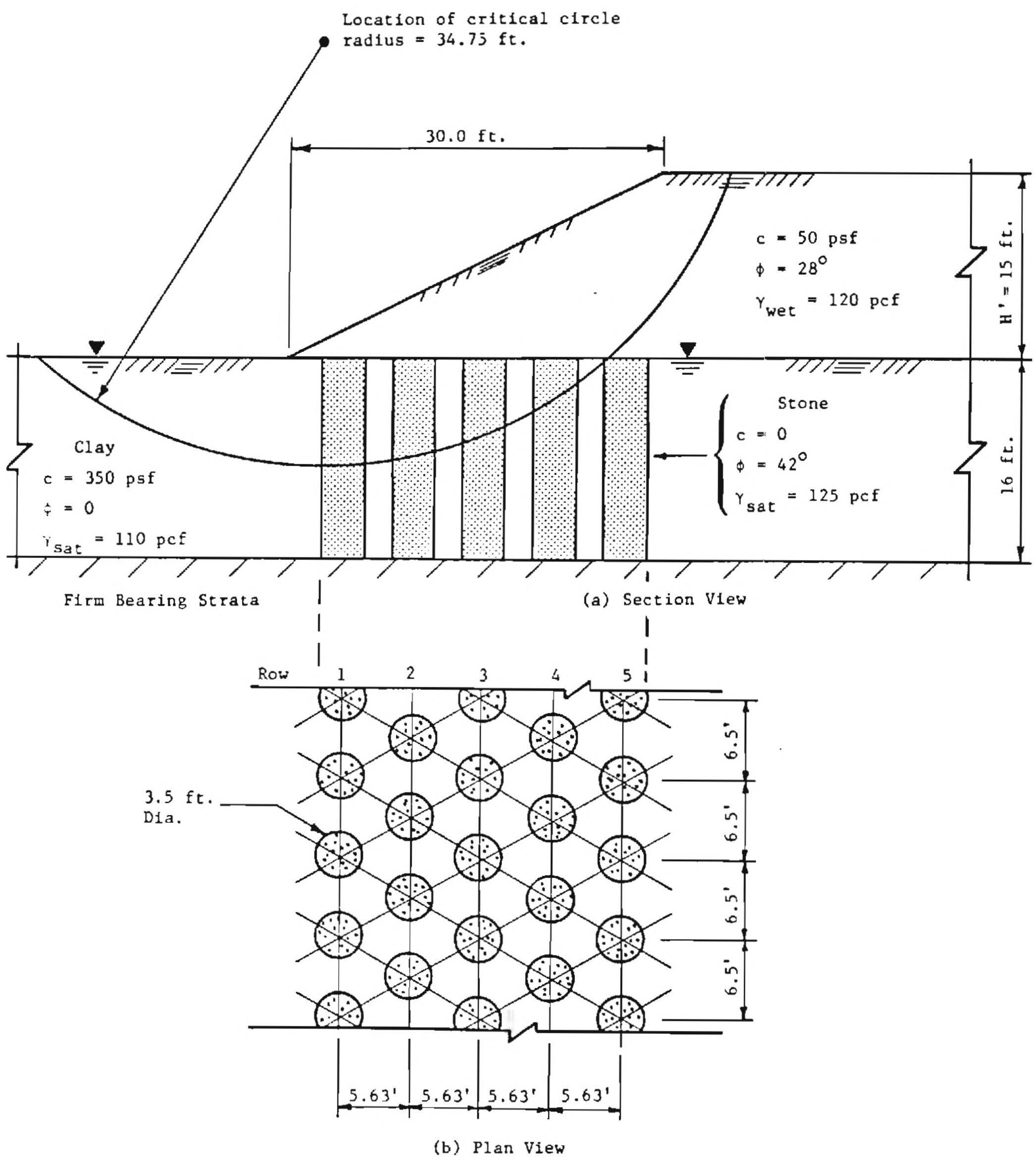

FIGURE 101. STABILITY EXAMPLE PROBLEM 1.

$A_s/(a_s s) = 9.62$ ft.$^2/(0.263 \times 6.5$ ft.$) = 5.63$ ft. which is the stone column spacing 0.866s in the direction perpendicular to the embankment length.

Now determine the characteristics of the fictitious strips that must be added to handle the effect of stress concentration in the stability analysis. Let the thickness $\tilde{T}$ of a fictitious strip be 0.3 ft. (91 mm) under the full embankment. Note that in this example no stone columns are actually used under the full embankment height for the first trial. However, stone column row 5 is located so that the edge of the tributary area is just at the break in the embankment. Therefore the unit weights calculated for the full embankment height can be used for each strip, with the thickness of the strips varying from zero at the toe to 0.3 ft. (91 mm) at the break (Fig. 102). An examination of equations (33) and (34) shows that this method gives the proper stress concentration in each strip. The thickness of the boundaries of each zone is calculated in Table 15.

The unit weights to use in the fictitious strips are calculated as follows:

$$\mu_s = n/[1 + (n-1)\, a_s] = 2.0/[1 + (0.263) = 1.58 \qquad (8b)^*$$

$$\mu_c = 1/[1 + (n-1)\, a_s] = 1.0/[1 + (0.263) = 0.792 \qquad (8a)^*$$

The correct unit weight to use above each stone column in the fictitious strip is

$$\gamma_f^s = (\mu_s - 1)\, \gamma_1 H'/\tilde{T} = (1.58 - 1)(120 \text{ pcf})\ 15 \text{ ft.}/0.3 \text{ ft.} \qquad (33)^*$$

$$= 3480 \text{ pcf}$$

and the unit weight to use above the soil in each fictitious strip is

$$\gamma_f^c = (\mu_c - 1)\, \gamma_1 H'/\tilde{T} = (0.792 - 1)\ 120 \text{ pcf } (15 \text{ ft.})/0.3 \text{ ft.} \qquad (34)^*$$

$$= -1248 \text{ pcf}$$

Material Properties and Zones are as follows (refer to Fig. 102):

Zone 1: $\gamma_w = 120$ pcf, $c = 50$ psf, $\phi = 28^o$

Zone 2: $\gamma_w = 0$, $c = 0$, $\phi = 0$

Zones 3,5,7,9,11,13: $\gamma = -1248$ pcf, $\phi = 0$, $c = 0$

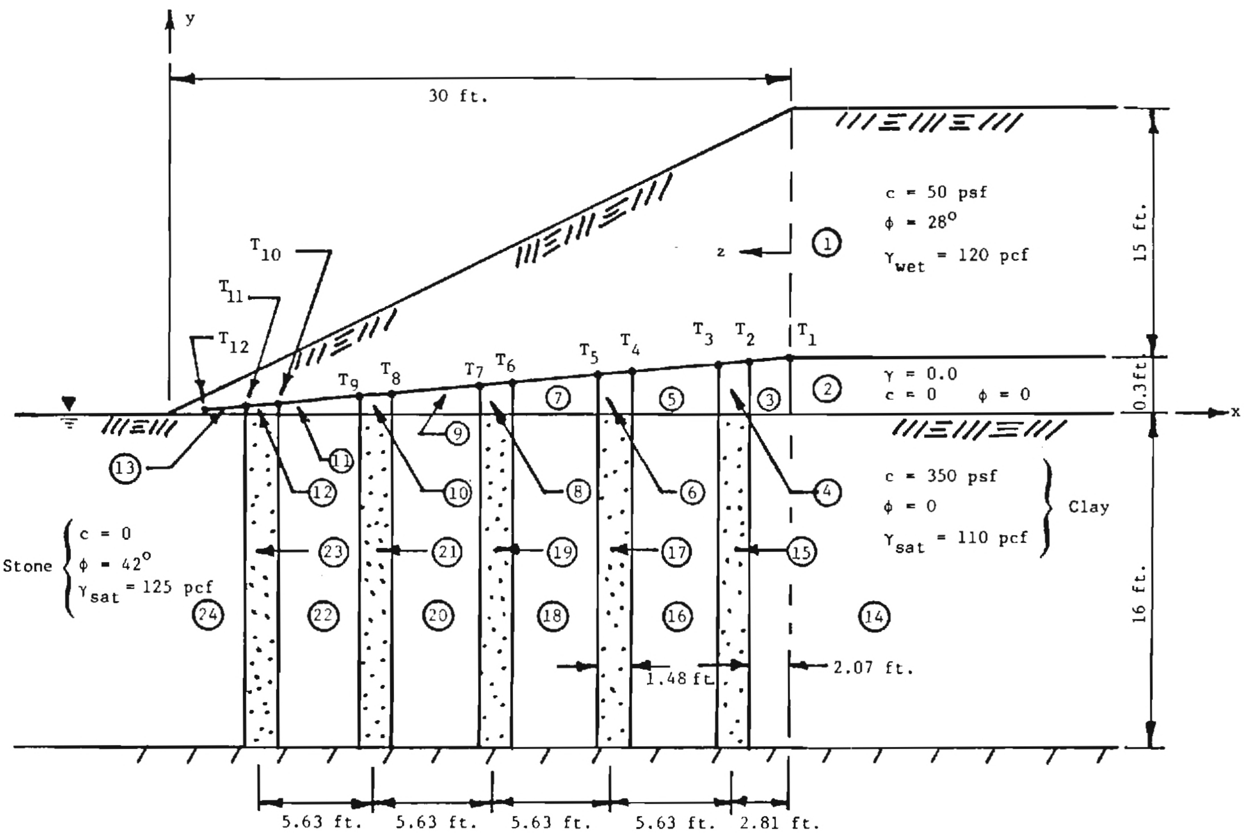

FIGURE 102. ZONES USED FOR COMPUTER IDEALIZATION FOR STABILITY EXAMPLE 1.

TABLE 15. THICKNESS OF FICTITIOUS STRIPS.

Location	z (ft.)	(30-z) (ft.)	Thickness, T_i, 0.01 (30-z) (ft.)
T_1	0	30	0.300
T_2	2.07	27.93	0.279
T_3	3.55	26.45	0.265
T_4	7.70	22.30	0.223
T_5	9.18	20.82	0.208
T_6	13.33	16.67	0.167
T_7	14.81	15.19	0.152
T_8	18.96	11.04	0.110
T_9	20.44	9.56	0.096
T_{10}	24.59	5.41	0.054
T_{11}	26.07	3.93	0.039
T_{12}	28.15	1.85	0.018

T_i = Thickness of Fictitious Layer at Location shown on Fig.102.

Zones 4,6,8,10,12: γ = +3480 pcf, ϕ = 0, c = 0

Zones 14,16,18,20,22,24: γ_{sat} = 110 pcf, ϕ = 0, c = 350 psf

Zones 15,17,19,21,23: γ_{sat} = 125 pcf, $\phi = 42^{o}$, c = 0

The calculated safety factor of the slope is shown in the table below for the following conditions: (1) no improvement with stone columns, (2) the stone column improvement shown in Fig. 101 using a stress concentration factor n=1, and (3) the same level of improvement with n=2.0 (the critical circle for this condition is shown in Fig. 101). A simplified Bishop analysis was performed using the GTICES Lease II computer program [122].

Case	n	Coordinate (1) x (ft.)	y (ft.)	R (ft.)	S.F.	Comment
1. No S.C.(2)	-	14.20	27.00	43.00	1.07	Base Failure
2. S.C.	1	2.90	26.00	42.00	1.38	Base Failure
3. S.C.	2.0	2.90	26.00	34.75	1.65	See Fig. 101

Notes: 1. Coordinates of critical circle (refer to Fig. 101 for location of x and y axes).

2. Notation: S.C. = stone column; S.F. = safety factor; R = radius of critical circle

APPENDIX F

RAMMED FRANKI STONE AND SAND COLUMNS

INTRODUCTION

Rammed stone and sand columns are constructed by the Franki Company primarily in Belgium and West Germany using a technique essentially the same as for the Franki pressure injected footing (concrete pile) [107,108]. Franki also constructs stone columns using a hydraulic vibrator following usual vibro-replacement construction procedures. The hydraulic vibrator used by Franki for vibro stone columns develops 111 H.P. (83 kw) and a centrifugal force of 38 tons (341 kN) at a frequency of 2500 rpm.

Franki rammed stone or sand columns are primarily used to support warehouses, including the floor slab, footings or slabs of low, multiple story buildings, tanks, tunnels through embankments, and stockpiles of raw materials. In these applications the main purpose is to limit total and differential settlement. Rammed stone columns are also sometimes used to increase the safety factor against sliding of a slope or to limit the horizontal soil displacement caused by surcharge loading by a raw material stockpile. This application reduces the passive bending pressure on piles supporting nearby structures.

The primary advantages of rammed stone columns over the conventional vibro method appears to be as follows: (1) The hole is cased using the Franki method and one method used by Datye, et al. [53]. As a result possible hole collapse is avoided in soft clays and cohesionless soils having a high groundwater table. Also stone is not dropped down an uncased hole. (2) Either sand or stone can be used with the Franki method. (3) Jetting and flushing water is not required. (4) Problems with soil erosion during flushing are avoided in organic soils and peat.

The main disadvantage of rammed stone columns is that the time required to construct the column is, for some applications, greater than for vibro methods. Also, driving the casing causes smear along the sides of the hole that reduces the radial permeability of the soil (refer to Fig. 56). Because of the high level of ramming used in the construction process, large excess pore pressures are created in the surrounding, low permeability cohesive soil. The excess pore pressures, however, reportedly dissipate rapidly resulting in rapid consolidation and strength gain. If a coarse, open graded stone is used, however, clogging may occur reducing the effectiveness of the stone column to act as a vertical drain. Also, for some applications such as slope stabilization, development of large excess pore pressures would be undesirable. A reduced level of energy input could be used to minimize this problem.

CASE HISTORIES

Tunnel Support

To limit settlement, a 13 ft. (4 m) wide tunnel was founded on four rows of rammed, coarse sand columns at Deinze, Belgium. The tunnel crosses through a 23 ft. (7 m) high compacted fill which applies a pressure of 2.5 ksf (125 kN/m^2) to the original soil at the base of the tunnel (Fig.103). The fill supports five railway lines. The load on the sand columns is mainly due to negative friction transmitted by the fill to the vertical exterior faces of the tunnel.

The sand columns have a 6.6 ft. (2 m) spacing on a square pattern. They are 2.1 ft. (0.64 m) in diameter and were constructed using a 17.7 in. (0.45 m) casing. The tips of the columns are founded in a dense sand layer at a depth of 57 ft. (17 m). Normally consolidated, loose silty sands of Quatenary age overly the dense sand. A stratum of stiff clay of Tertiary age is found beneath the dense sand. The measured cone resistance before construction is shown in Fig. 103 as a function of depth.

Composite Gravel-Concrete Column

Composite rammed Franki gravel and concrete columns were constructed in 1976 to 1977 at the Beaver Valley Nuclear Station at Shippingport, Pennsylvania. The columns were used to densify a loose sand layer susceptible to liquefaction during an earthquake. The loose sand layer is located from a depth of 35 to 80 ft. (10.7-24.4 m) below the surface. Dense coarse sands and gravels overlay the loose sand.

The columns were constructed with a 21 in. (0.535 m) diameter casing using a 7.5 ft. (2.28 m) spacing in a triangular pattern. The columns were constructed through the loose sand stratum by ramming successive expanded bases or dry, lean concrete using a 3 ft. (0.9 m) vertical driving interval. Sand and gravel shafts were used above in the dense sands and gravels.

In the loose zone successive bases were built using 140,000 ft-lb blows (1.9 MN-m) up to a specified number of blows per 5 ft.^{3} (0.14 m^3). The consumption of dry concrete was about 15 ft.^{3} (0.42 m^3) per base. Although the primary purpose of the columns was to densify the loose sand, use of concrete to construct these columns also stiffened the stratum.

CONSTRUCTION

Rammed Franki stone or sand columns are generally constructed using 16, 18, 20.5 and 24 in. (400-600 mm) diameter casing. The casing is driven to the specified depth, usually by hammering on a temporary stone or sand plug located at the bottom of the casing. A 3 to 4.5 ton ram is used in construction, which is the same as for the Franki concrete pile. The height of fall, usually 13 to 20 ft. (4-6 m), is chosen considering the soil strength

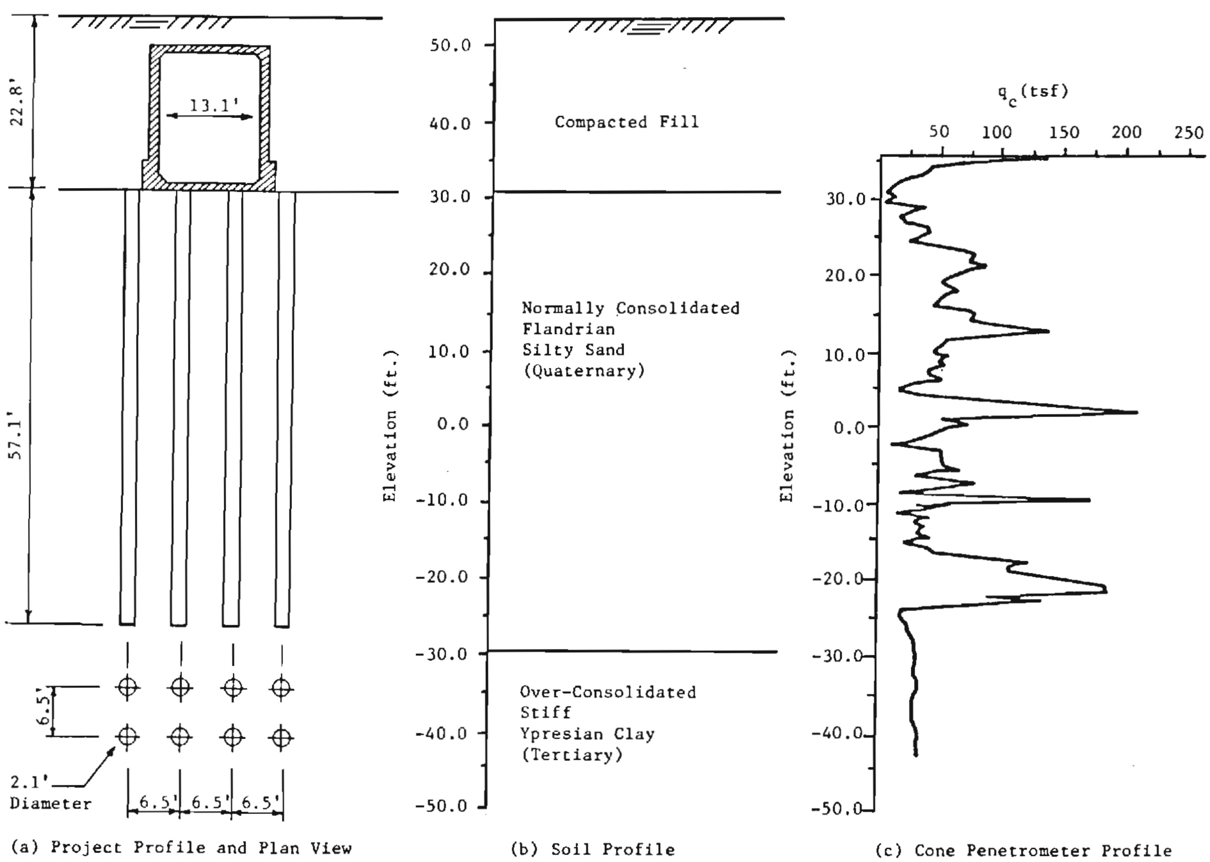

FIGURE 103. USE OF RAMMED STONE COLUMNS FOR TUNNEL SUPPORT AT DEINZE, BELGIUM.

and project requirements.

When the specified depth is reached, the plug is driven out by hammering with the casing maintained in position or slightly pulled up by tension ropes. The stone or sand column is constructed by ramming about 3.5 $ft.^3$ (100 liters) of material in successive lifts as the column is brought up. As the column is constructed, the casing is progressively extracted. The diameter of the compacted column is determined by the effective volume of material used per unit column length. The density of the granular column increases as the completed diameter of the column, compared to the casing, increases. The increase in diameter is usually between 4 in. (10 cm) and 12 in. (30 cm). Columns having a 36 in. (90 cm) diameter are routinely constructed with a 24 in. (60 cm) casing, and 32 in. (80 cm) diameter columns with a 20.5 in. (52 cm) casing. Since the soil to be improved with stone or sand columns in general is not very stiff, columns can be constructed with diameters greater than 48 in. (1.2 m). Franki has constructed concrete bases up to 60 in. (1.5 m) in diameter in both loose sandy and stiff clayey soils.

Depending on the soil strength profile, the following three methods can be used to construct Franki rammed columns.

1. The easiest and most certain technique is to construct a column with a constant diameter. This method requires varying the energy applied depending upon the soil strength.

2. A second approach is to increase the volume of material over the required volume as long as a critical value of energy per unit length of column has not been developed. As a result, the column diameter changes with the soil strength. The problem with this method is in determining the required critical compaction energy level. The best approach is to use field tests comparing profiles of, for example, measured cone resistance, and the energy per unit length required to develop columns of various diameters. To avoid problems a certain minimum length of plug should be maintained throughout driving. This minimum plug length must be the largest necessary length for any stratum to prevent soil or water from entering into the casing. Sometimes this requirement is expensive, and the benefit resulting from the diameter of the column changing with the soil strength is lost compared to a column having a uniform diameter; the diameter of a uniform column would be the largest necessary to limit settlement to an acceptable level.

3. A practical alternative which is easier to construct and more reliable than the constant energy method (Alternative 2) is to develop a stepped diameter column. Consider when a column having an insufficient diameter is formed through a soft layer. Following the stepped column approach, the casing is plugged and redriven through the completed column to the bottom of the soft layer. A second column, lying on the axis of the first, is then constructed upward to obtain the required diameter in that stratum.

Using experience gained with zero slump concrete Franki piles, the volume of rammed stone or sand columns is assumed for preliminary estimates to be 0.8 the loose volume. During construction of the column, the length of the granular plug remaining in the bottom of the casing is never allowed to get small enough to let soil or water to enter the casing. Stone columns are generally constructed using rounded material having a maximum size of 1.2 to 2.4 in. (30-60 mm). When necessary, as for concrete piles, the base of the column can be enlarged.

With the available equipment and customary stone column diameters used, a production rate of 400 to 660 ft. (120-200 m) of column is usually obtained per 8 hour shift, depending on the soil strength.

DESIGN

Franki stone columns are always carried down to firm bearing material. Design column loads vary from 20 to 60 tons. A stone column spacing of 3 diameters is usually employed with the minimum being 2.5 diameters. For coarse stone an angle of internal friction up to 45° to 50° is used for stability analyses. For sand an angle of internal friction less than 40° is used. The larger values of the angle of internal friction requires a well densified material. A column is considered to be well densified if the diameter of the column is equal to or greater than the diameter of the casing plus 4 in. (10 cm).

The deformation and the bulging load of a single column are estimated from Menard pressuremeter test results, or from Vesic cavity expansion theory (refer to Chapter III). For large groups of columns, the latter theory is modified to take into account the fact that the radius of influence of the columns is limited, and the vertical stresses are increased by the surcharge transmitted at the soil surface. A coefficient of at-rest earth pressure K_o less than one is used along the zone of influence.

Two different type settlement analyses are made when (1) the stiffness of the slab transmitting the loads does not permit differential settlement to occur between the column and the soil (equal strain assumption), and (2) the transmitting element is flexible, i.e., the settlement of the soil is larger than settlement of the columns. The modulus of elasticity of the soil is obtained from the results of laboratory consolidation tests, cone penetration tests, or pressuremeter tests. Drained soil response is used to consider long-term effects.

For columns loaded between about 20 and 30 tons, the settlement is usually between approximately 0.4 and 0.8 in. (10-20 mm). In low permeability soils where high excess pore pressures are anticipated, a sand is usually preferred to prevent clogging. Clogging would reduce the rate of water flow vertically through the column resulting in a greater length of time for the soil to consolidate and gain strength. Sand, which is also used when gravel is not available or is too expensive, is believed to result in more settlement than stone [51.]

FIELD INSPECTION

When the required diameter is constant over the length of the column, field inspection is conducted by estimating the diameter of the stone or sand column. Therefore, the quantity of material consumed during construction is measured per unit of length of column, and the diameter calculated. When a critical energy per unit of length is specified in addition to a minimum diameter, both the quantity of material added and the energy used are measured per unit of column length.