Intern rapport nr. 1933

Jordnagling - Stipendreise til Seattle og Portland 1996

Februar 1997

Veglaboratoriet

Intern rapport nr. 1933

Jordnagling - Stipendreise til Seattle og Portland 1996

Sammendrag

Rapporten beskriver en stipendreise til Seattle og Portland i november 1996 for å studere jordnagling.

En rekke jordnaglingsprosjekter er beskrevet. Metoden brukes i utstrakt grad i staten Washington og Oregon.

Befaringer og møter med Washington State Department of Transportation, Oregon State Department of Transportation, Federal Highway Administration og lokale konsulenter er beskrevet.

Erfaringene viser at jordnagling er en teknisk og økonomisk gunstig metode hvor forholdene ligger til rette for det.

En omfattende "Manual for design and construction monitoring of soil nail walls" fra Federal Highway Administration ble ferdig i november 1996.

Emneord: *Jordnagling, instrumentering, retningslinjer, korrosjonsbeskyttelse*

Seksjon: *3520 - Geologi- og geotekniokkontoret*
Saksbehandler: *Jan Vaslestad* */BN*

Dato: *Februar 1997*

Statens vegvesen, Vegdirektoratet
Veglaboratoriet
Postboks 8142 Dep, 0033 Oslo
Telefon: 22 07 39 00 Telefax: 22 07 34 44

Innhold

1. Innledning

Undertegnede fikk stipend fra Vegdirektoratet for å studere jordnagling i 1996.

Opprinnelig var planen å reise til Tyskland og Frankrike.

Etter et innledende litteraturstudium kom det raskt frem at amerikanerne nå har kommet langt frem når det gjelder utvikling av metoder, og Federal Highway Administration (FHWA) har skrevet retningslinjer og veiledninger, FHWA (1990, 1991, 1993 og 1994). Se referanseliste kap. 7.

I tillegg tok jeg kontakt med Stan Boyle som i 1996 var stipendiat ved Norges Geotekniske Institutt. Stan Boyle tok doktorgraden sin ved University og Washington i Seattle på temaet armert jord. Han satte meg i kontakt med Professor Bob Holtz ved University of Washington. Jeg fikk et veldig hyggelig og informativt brev fra Bob Holtz, som også har en rekke kontakter i Washington State Department of Transport (WSDOT), FHWA og lokale konsulenter som har erfaring med jordnagling.

Bob Holtz informerte meg også om Valle Scholarship and Scandinavian Exchange Program, hvor Dr. Arild Eikum ved Aquateam i Oslo er norsk kontakt (se vedlegg 1).

Denne positive kontakten medførte at Seattle og Portland ble valgt som studiemål istedenfor Tyskland og Frankrike. Disse to landene ble dessuten gjenstand for en studietur med stipend fra NVF i 1989, Oset og Aabøe (1989).

Dessuten ble det skrevet en omfattende rapport om "state of the art" for jordnagling i Europa etter en såkalt "Scanning tour" utført av en amerikansk gruppe fagfolk interessert i jordnagling, FHWA (1993a).

De franske retningslinjene Clouterre ble også oversatt til engelsk etter denne studieturen, FHWA (1993b).

Etter en relativt omfattende kontakt pr. telefax med de aktuelle kontaktpersoner ble følgende program lagt opp:

- *Mandag 4. november:* University of Washington
- *Tirsdag 5. november:* Geo Engineers (Geoteknisk konsulentfirma)
- *Onsdg 6. november:* Washington State Department of Transportation
- *Torsdag 7. november:* Oregon Department of Transportation og Federal Highway Administration
- *Fredag 8. november:* Golder Associates (Geoteknisk konsulentfirma)

Forøvrig bodde jeg på et lite koselig hotell (rimelig) med meget god service og hyggelig betjening. Hotellet het University Inn og lå i University district i gangavstand fra Universitetet. Adresse etc. er vist i vedlegg 2.

For evenetuell kontakt og oppfølging med de personene jeg møtte, ligger kopi av visittkortene i vedlegg 3.

2. University of Washington, Seattle (*Mandag 4. november*)

Jeg møtte Professor Bob Holtz på kontoret hans på universitetet.

Bob Holtz har i en årrekke vært svært aktiv innenfor området armert jord og geosynteter og har forfattet en rekke lærebøker og publikasjoner. Dagen ble brukt til omvisning i laboratoriene og diskusjoner med forskere og forelesere i geoteknikk.

Spesielt interessante diskusjoner hadde jeg med doktorgradstudenten Wei Lee fra Taiwan.

Etter ønske fra professor Bob Holtz, hadde jeg forberedt et seminar med tittelen "Case histories of soil nailing and reinforced earth in Norway" (se vedlegg 4).

Dette var svært nyttig, for i tillegg til studenter og forelesere, var det også til stede en rekke konsulenter og entreprenører fra Seattle-området.

Vi fikk til en svært interessant diskusjon, og de fleste av de personene jeg skulle treffe i løpet av uka var til stede.

University of Washington er et vakkert universitet med mange fine bygninger, grønne parker og fontener, se bilde 1.

Bilde 1: University of Washington i Seattle

Professor Bob Holtz var forøvrig en svært hyggelig og sosial person.

På mandag kveld ble jeg invitert med Bob og hans kone på konsert med Seattle Symfoniorkester. Etter en interessant konsert spiste vi nydelig fisk på en av Seattle's gode fiskerestauranter.

3. Jordnaglingsprosjekter i Seattle og Oregon, befaring med Geo Engineers (*Tirsdag 5. november*)

Tidlig på morgenen ble jeg hentet av Gordon Denby, som er en av lederne i konsulentfirmaet Geo Engineers som har spesialisert seg på jordnagling. I tillegg møtte vi Ted Robinson som er Vice President i Malcolm Drilling Co., Inc. Dette er en av de største borefirmaene i USA, og utfører en rekke jordnaglingsjobber.

Det første prosjektet vi så på var nybygg på Microsoft-"campus". Dette var et stort område med mye bygging. På veien passerte vi også tomta til Bill Gates (Grunnleggeren av Microsoft) hvor det pågikk anleggsarbeider, blant annet jordnagling.

På dette anlegget ble jordnagling brukt ved en 10 m utgraving under bakken.

Jordnaglene bestod av injiserte 25 mm galvaniserte kamstål med 6 m lengde. En detalj av naglehodet med en spesiell utformet stålplate med 4 påsveisede bolter som blir støpt fast i den utenpåliggende 10 cm tykke plasstøpte betongveggen er vist på bilde 2.

Bilde 2: Gordon Denby viser stålplaten montert på naglehodet

Dette var en vanlig utførelse av naglehodet på jordnaglingsprosjekter i USA.

På bilde 3 er det vist en detalj av dreneringen bak muren.

Bilde 3: Drensstriper av geosyntet

Drensstripene var 30 cm brede og ble lagt horisontalt og vertikalt med avstand 1 m.

Malcolm Drilling hadde utviklet en spesialborrigg for jordnagling, se bilde 4.

Bilde 4: Spesialborrigg for jordnagling

Det neste prosjektet var utvidelse av en motorveg med ett felt under en bro. Dette er en mye brukt metode hvor det er behov for ett ekstra felt inn mot et brufundament.

En typisk prinsippskisse av en slik utvidelse er vist på fig. 1.

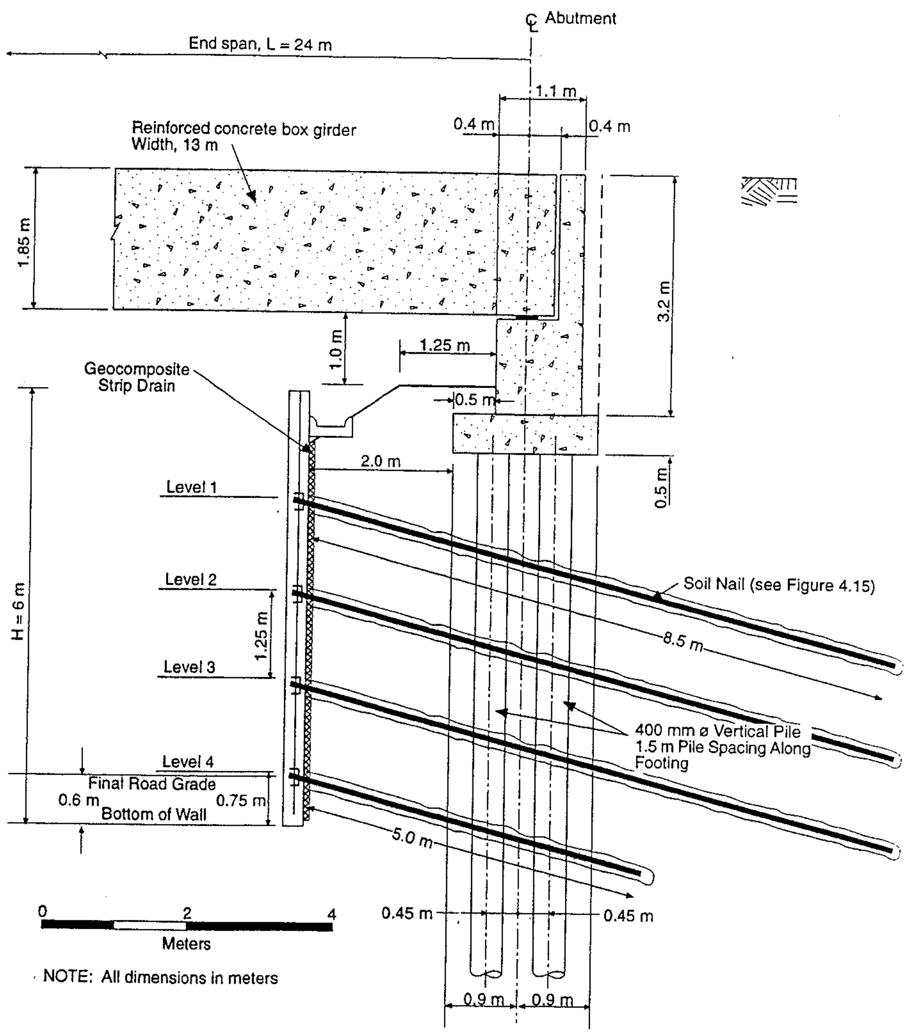


Fig. 1. Jordnagling brukt ved utvidelse med ett felt inn mot et brufundament (fra FHWA, 1996)

På bilde 5 er vist ferdig sprøytebetongoverflate før den endelige plasstøpte betongveggen med tykkelse 15 cm skal støpes.

Bilde 5: Jordnagling brukt ved utvidelse med ett felt under en bru

På bilde 6 er vist detaljer av naglehode.

Bilde 6: Jordnaglingsmur med naglehoder før endelig plasstøpt betongvegg bygges utenpå

Detaljer angående utførelse av jordnagler, drenering og frost er vist i vedlegg 5 i denne rapporten.

Det siste prosjektet vi så på denne dagen var bygging av nytt konserthus i Seattle sentrum. Her ble det brukt en kombinasjon av jordnagling og berlinervegg, se bilde 7.

Bilde 7: Jordnagling og berlinervegg brukt ved bygging av nytt konserthus i Seattle sentrum

4. Befaring og møte med Washington State Department of Transportation (*Onsdag 6. november*)

WSDOT holder til i Olympia, ca. 1 time kjøring fra Seattle. På tross av dette ble jeg hentet på hotellet av Tony Allen (State Geotechnical Engineer) som var leder for geoteknikk-avdelingen i WSDOT. Robert Kimmerling (Chief Foundation Engineer) og ekspert på jordnagling var også med.

Både Tony og Robert var på tross av sine ansvarsfulle stillinger, relativt unge og veldig kunnskapsrike. De var dessuten veldig åpne og Tony og jeg har allerede utvekslet en del veiledninger og retningslinjer og fått til et fint samarbeid som kan være svært nyttig fremover.

Det første prosjektet vi så på var en skjæring hvor det var brukt jordnagling i en støttekonstruksjon med avtrapping, se bilde 8.

Bilde 8: Støttekonstruksjon utført med jordnagling og plasstøpt betongfront

Bilde 9 er tatt på en overgangsbru på dette prosjektet. Brua var forøvrig full av sprekker og riss som også kan sees på bildet.

Bilde 9: Robert Kimmerling til venstre og Tony Allen til høyre. Begge fra WSDOT.

Dessverre var det ingen prosjekter under utførelse i området rundt Seattle på dette tidspunktet, men vi fikk sett en rekke ferdige prosjekter, se bilde 10.

Bilde 10: Jordnaglingsmur under bru (utvidet med ett felt)

Bilde 11: Jordnaglingsmur i terrasser

Bilde 12: Støttekonstruksjon med jordnagling

Bilde 13: Jordnagling brukt på begge sider i en skjæring

Bilde 14: Støttekonstruksjon utført med jordnagling

På denne turen så vi også et par jordarmeringskonstruksjoner.

Det første prosjektet var et landkar bygget opp med oppbrettsløsning med geotekstil. Konstruksjonen hadde to formål: Forbelastning av dårlig grunn inn mot landkaret (bratt skråning) og en minimalisering av horisontalt jordtrykk mot landkaret, bilde 15. Bilde 16 viser den samme brua under bygging.

Bilde 15: Armert jord inn mot landkar for forbelastning og reduksjon av jordtrykk

Bilde 16: Bru under bygging

Vi så også en annen type armert jord med front av stålnett (Welded-wire-systemet), bilde 17, 18 og 19.

Bilde 17: Armert jord inn mot landkar

Bilde 18: Armert jord konstruksjon bygd med "Welded-wire" systemet

Bilde 19: Armert jord, utsparing for gatelys

5. Befaring og møte med Oregon Department of Transportation og Federal Highway Administration (*Torsdag 7. november*)

Ronald Chassie (Regional Geotechnical Engineer, Region 10) er den personen i Federal Highway Administration som har mest erfaring og kompetanse på jordnagling. Han har kontor i Portland i Oregon, ca. 4 timers biltur fra Seattle.

Jeg ordnet med leie av bil på onsdag kveld og tidlig torsdag morgen reiste jeg fra Seattle. Fra hotellet mitt i University district rett ved University of Washington var det enkelt å finne frem til Interstate 5, som er hovedveien fra Seattle til Oregon.

I tillegg til Ron Chassie møtte jeg også Claude Sakr (Project Manager) og Jan Six (Foundation Design Engineer). Disse to utgjør Oregon DOT's kompetanse på jordnagling, så dette så ut til å bli en lovende dag. Bilde 20 viser et foto av jordnaglingsekspertene foran et ferdig jordnaglingsprosjekt.

Bilde 20: Fra venstre: Ron Chassie (FHWA), Claude Sakr og Jan Six (Oregon DOT) foran et jordnaglingsprosjekt i Portland

Dette var et prosjekt hvor det hadde oppstått en del problemer i anleggsfasen på grunn av ensgradert sand. Dette medførte at utgravingen ikke er stabil lenge nok til jordnaglene kan etableres. Tiltak mot dette er å la det stå igjen en liten voll eller gå på med sprøytebetong umiddelbart eter utgraving. Disse metodene er detaljert beskrevet i FHWA (1994).

Det neste prosjektet vi så på var utvidelse under et brufundament for en bru på Interstate 5 (Pacific Highway). Dette prosjektet er grundig instrumentert og beskrevet i referansene Dumas (1991) og Sakr and Kimmerling (1995).

Bilde 21 viser Claude Sakr som var ansvarlig for instrumentering av prosjektet foran et instrumentert snitt.

Bilde 21: Claude Sakr ved et instrumentert snitt på jordnaglingsprosjektet på Interstate 5 i Portland

Utgravingen var totalt 6 m høy, og det ble brukt 43 mm epoxy-belagt kamstål i 125 mm borhull som ble injisert. Jordnaglene var opptil 7,2 m lange.

Et tverrsnitt gjennom brufundamentet er vist på fig. 2.

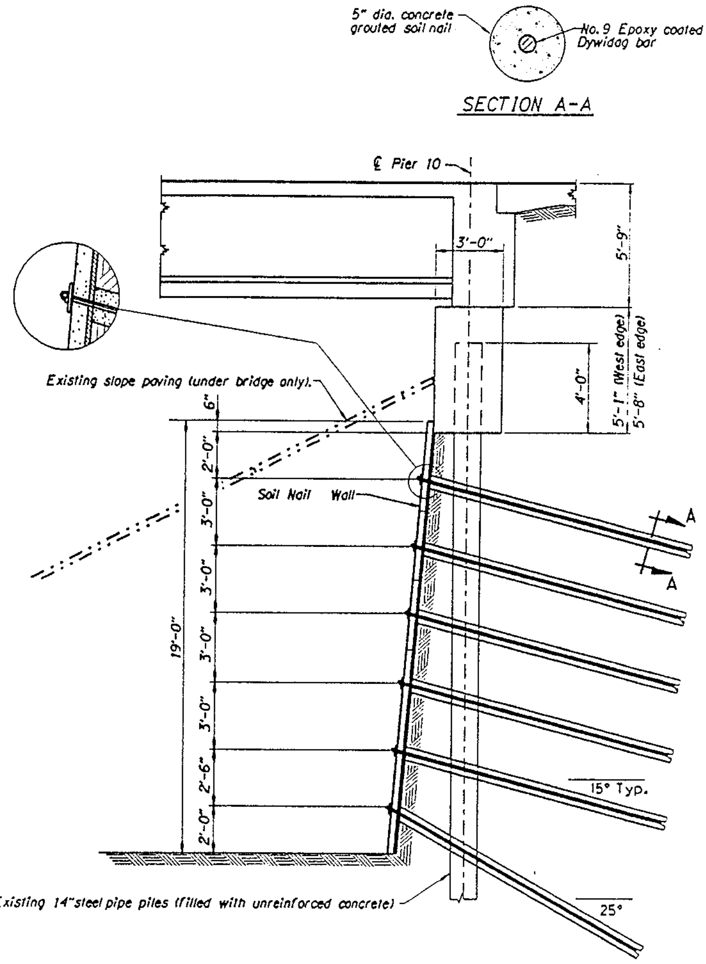


Fig. 2. Tverrsnitt av brufundament med jordnaglingsmur

Brua var fundamentert på 35 cm diameter stålrør fylt med betong, og jordnaglene ble installert mellom pelene.

Totalt ble det påført et 200 mm tykt lag med sprøytebetong i to lag (90 + 110 mm).

Et av de to instrumenterte snittene er vist på figur 3.

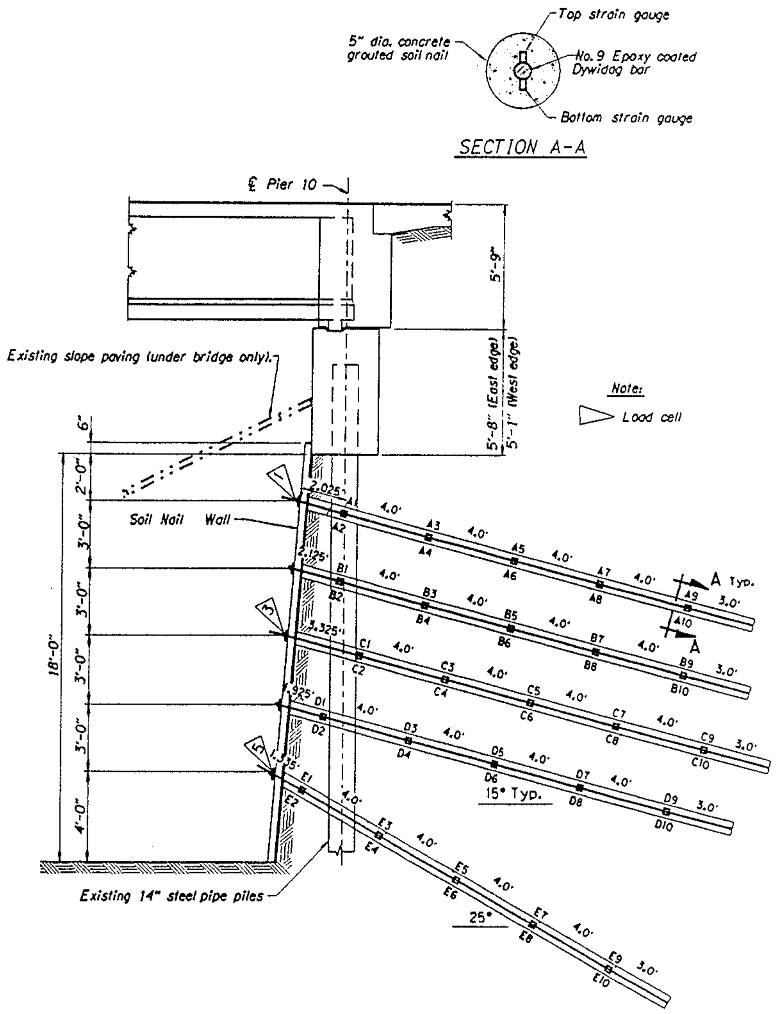


Fig. 3. Instrumentert snitt

På bilde 22 kan sees ferdig konstruksjon med måleskap for avlesning av instrumenter på toppen.

Bilde 22: Ferdig konstruksjon med måleskap på toppen

Bilde 23: Detalj av "monitoring well" for tilgang til lastcellen

Det siste prosjektet vi så på i Portland var utbygging av 29 km forlengelse av togsystemet (light rail). Her var det brukt jordnagling i stor utstrekning i tildels høye skjæringer, opp til 12 m høyde.

På bilde 24 er vist lange og høye murer utført med jordnagling.

Bilde 24: Jordnaglingsmurer i skjæring for Portland Light Rail-systemet

Bilde 25: Den høyeste muren (12 m)

Bilde 26: Instrumentert jordnaglingsmur foran brulandkar

Bilde 27: Jordnagling i skjæring foran inngang til "cut and cover" tunnel

Bilde 28: Mur i 2 høyder, på nedre del er det brukt jordnagling, på øvre del armert jord

Ron Chassie har i flere år arbeidet med en "Manual for design and construction monitoring of soil nail walls". Manualen var ferdig akkurat denne første uken i november 96, og jeg var så heldig at jeg fikk med en kopi (468 sider), FHWA (1996). Manualen er registrert i biblioteket på Veglaboratoriet.

Noen sentrale punkter fra manualen er vist i vedlegg 5.

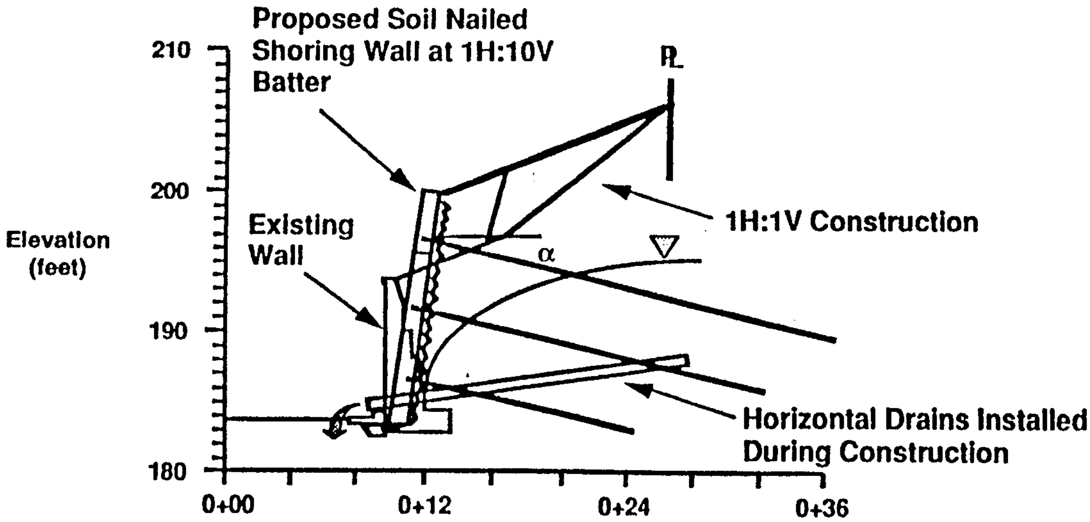
Manualen er utarbeidet av konsulentfirmaet Golder Associates Inc.

På fredag morgen har jeg en avtale med en av forfatterne av manualen fra Golder Associates: David Cotton.

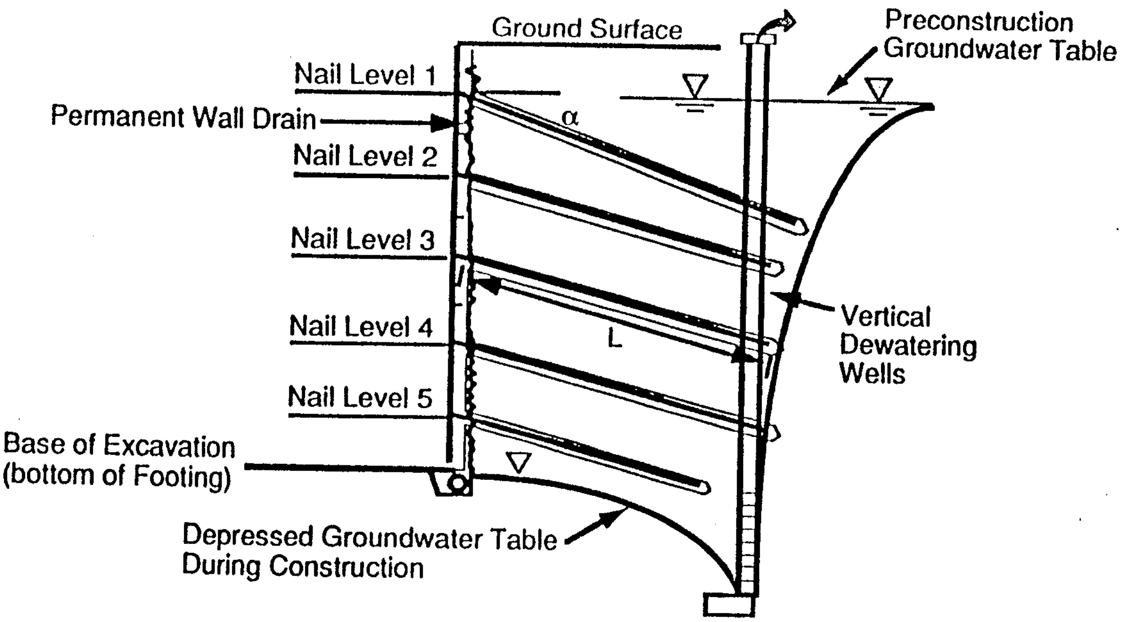
6. Møte med Golder Associates Inc. *(Fredag 7. november)*

David Cotton (Principal) kom til hotellet mitt på fredag morgen. Han hadde vært til stede på seminaret på mandag, og vi diskuterte blant annet problemstillinger vi hadde i forbindelse med utførelse av jordnaglingsprosjekter i Norge, Vaslestad (1996).

I forbindelse med høy grunnvannstand skisserte David Cotton to forskjellige løsninger: Horisontale dren (figur 4) og vertikale pumpebrønner (figur 5).



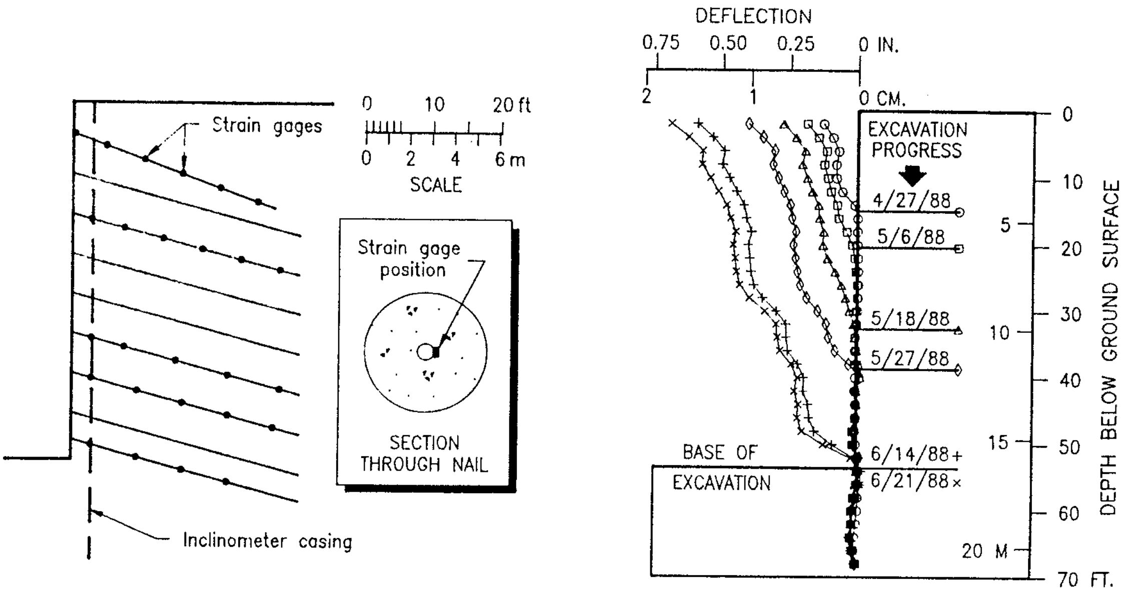
Figur 4: Horisontale dren



Figur 5: Vertikal pumpebrønn

I Seattle ble det i 1989 bygd en 16,8 m dyp utgraving som ble utført med jordnagling. Dette var den første permanente jordnaglingskonstruksjonen utført i USA.

Konstruksjonen er vist på figur 6.



Instrumented Section **Wall Deflections**

Figur 6: Jordnagling brukt i forbindelse med dyputgraving i Seattle

Utførelse og resultater fra instrumentering er beskrevet av Thompson and Miller (1990), begge fra Golder Associates.

I forbindelse med utarbeidelse av jordnaglingsmanualen fra FHWA skal Golder Associates holde en rekke seminarer rundt i USA. Vi diskuterte muligheten for å arrangere et slikt seminar i Norge.

Dette kunne for eksempel arrangeres ved Veglaboratoriet med deltagere også fra Sverige og Finland. Etter at jeg kom tilbake til Norge fikk jeg en telefax fra John Byrne hvor Golder tilbyr seg å arrangere et slikt seminar. Golder Associates har også laget et dimensjoneringsprogram Gold Nail, som de nylig har solgt til FHWA. Kostnaden for dette programmet er $ 1000 per lisens.

For eventuell oppfølging av disse kontaktene, ligger kopi av visittkortene til de personer jeg traff i vedlegg 3.

På fredag kveld ble jeg invitert hjem til Stan Boyle og hans kone Lynda som så var tilbake i Seattle etter endt opphold i Norge.

Lynda er utdannet bygningsingeniør og hadde nettopp startet et eget konsulentfirma. Hun trengte oppdrag og kopi av visittkortet ligger i vedlegg 3.

Totalt sett var det en meget utbytterik studietur, både faglig og sosialt.

På avreisedagen søndag var det også mulig å få et glimt av Mount Rainier i nydelig høstvær, se bilde 29.

Bilde 29: Mount Rainier sett fra flyplassen i Seattle

7. Referanser

Dumas C. (1991)
Permanent soil nail wall, I-5, Portland Oregon, Report Oregon DOT

Federal Highway Administration (1990)
Reinforced Soil Structures Volume 1. Design and Construction Guidelines, Chapter 6.
Design of nailed soil retaining structures, pp. 187-295. Publication No FHWA-RD-89-043

Federal Highway Administration (1991)
Soil nailing for stabilization of highway slopes and excavations.
Publication No FHWA-RD-89-193

Federal Highway Administration (1993a)
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Recommandations Clouterre 1991. Soil Nailing Recommendations (English translation)

Federal Highway Administration (1994)
Soil nailing - Field inspectors manual. Soil nail walls.
Publication No FHWA-5A-93-068

Federal Highway Administration (1996)
Manual for Design and Construction Monitoring of Soil Nail Walls.
Publication No FHWA-SA-96-069

Oset, F. og Aabøe, R. (1989)
NVF-stipend: Jordnagling. Intern rapport 1425 fra Veglaboratoriet.
Oslo Desember 1989

Sakr, C. and Kimmerling R. (1995)
Soil nailing of a bridge embankment. Report 2: Design and field performance report. Project OR 89-07. Oregon DOT and FHWA

Thompson S. R. and Miller I. R. (1990)
Design, construction and performance of a soil nailed wall in Seattle, Washington. Geotechnical Special Publication No. 25, ASCE, pp. 629-643

Vaslestad J. (1996)
Jordnagling - Foredrag på kursdagene NTNU 1996. Intern rapport 1892 fra Veglaboratoriet

Vedlegg 1

UNIVERSITY OF WASHINGTON
SEATTLE, WASHINGTON

Valle Scholarship and Scandinavian Exchange Program
102 Wilson Laboratory
Office of the Director

September 12, 1996

Jan Vaslestad, Dr. Ing., Senior Research Engineer
Road Research Laboratory, Geology and Soil Mechanics Division
Gustadalleen 25, PO Box 8142
Dep N-0033, Oslo, Norway

Dear Dr. Vaslestad:

Professor Robert Holtz at the University of Washington has suggested that you might be interested in receiving information regarding the Valle Scholarship and Scandinavian Exchange Program. Thus I am enclosing a small brochure and newsletter for the Program which may be useful to you. If you would like to discuss more details of Valle Program, we would be happy to hear from you. Alternatively, you could contact Dr. Arild Eikum at Aquateam, Bertrand Narvesens vei 2, Oslo, telephone 22 67 93 10. Arild is an affiliate professor at the University of Washington and is our program representative in Norway.

As you will note, the purpose of the program is to promote exchange of graduate students between the University of Washington and programs in the Nordic countries. The field of study is restricted to Civil Engineering and closely related fields such as Architecture. Normally, students coming from Scandinavia have pursued a master's degree at the University of Washington. Other government research groups such as NIVA have had staff who have been Valle scholars.

Your interest in the Valle Program is appreciated.

With best wishes,

Dale A. Carlson, Director

cc: Professor Robert Holtz
    Professor Arild Eikum

encl.

Mailing Address:
Valle Scholarship and Scandinavian Exchange Program
University of Washington
Box 352130
Seattle, Washington 98195-2130

Telephone: (206) 685-2312
Fax: (206) 543-2907
e-mail: valle@rio.engr.washington.edu

the

VALLE SCHOLARSHIP

and

SCANDINAVIAN EXCHANGE PROGRAM

at the

UNIVERSITY of WASHINGTON

Graduate Exchange Scholarships with Institutions in Scandinavia

and

Graduate Scholarships at the University of Washington

Academic Year 1997-1998

The College of Engineering and
The College of Architecture and Urban Planning
University of Washington
Seattle, Washington

Fields include:

- Civil Engineering
  - Environmental Engineering & Science
  - Structural, Geotechnical Engineering & Mechanics
  - Transportation, Surveying & Construction Engineering
- Architecture
- Landscape Architecture
- Urban Design & Planning

Valle Program Office Personnel

Dale A. Carlson, Director and Professor
John F. Ferguson, Associate Director and
Professor and Chair, Civil Engineering
Bobbie Nelson Greer, Administrative Assistant
Tel: (206) 685-2312
Fax: (206) 543-2907
e-mail: valle@rio.engr.washington.edu

For further information please contact the Valle office or mail the form below to:

Director
Valle Scholarship and Scandinavian Exchange Program
University of Washington
Box 352130
Seattle, Washington 98195-2130 USA
or read about us on the World Wide Web at:
http://www.engr.washington.edu/activities/valle/

Information Requested			
Your Name			
Organization			
Office/Street Address			
City	State	ZIP Code	Country

The Valle Program

History

Henrik Valle was born in Os, Norway in 1896. He received his education at Norges Tekniske Høgskole in Trondheim, where he graduated in civil engineering and military science.

Valle emigrated to the United States in 1925. Settling in Seattle, he joined the Peder P. Gjerde Construction Company. He remained with the company until he assumed ownership in 1936 and renamed it the Henrik Valle Company.

He was active in the construction of landmark building projects throughout the greater Seattle area and in Alaska. Projects on the University of Washington campus include the Student Union Building, Suzzallo Library addition, and Johnson and Physics Halls. In their wills, Henrik and Ellen Stray Valle bequeathed funds to underwrite the Valle Program

From its inception in 1980 and through 1996, the Valle Scholarship and Scandinavian Exchange Program has funded over 360 students.

Exchange Scholar Selection

Scholars are chosen from applicants who will be pursuing graduate degrees in the Department of Civil Engineering or closely related fields such as Architecture, and who have been admitted to the Graduate School at the University of Washington. The first professional degree must have been attained prior to the beginning date of the scholarship. Applicants must be citizens or permanent residents of the United States or of Denmark, Finland, Iceland, Norway or Sweden. Proficiency in English is required for students attending the University of Washington. Application forms are available from the Director, Valle Scholarship and Scandinavian Exchange Program.

Applications must be received by February 1 to be considered for the following academic year. Applicants should note that a research topic and plan of study should be developed and submitted as part of the application. That plan must show how the exchange period will integrate into previous and proposed work at the home institution. It is the responsibility of the applicant to show that linkages exist for the proposed research between the home institution and the exchange institution. Thus, persons interested in applying should begin the application process several months before the February 1 deadline.

Applicants are welcome to telephone, write, or e-mail the Program Director regarding their questions and concerns.

Award

The award includes a monthly stipend approximately equivalent to a graduate assistantship and provides for resident tuition.

The scholarship award period varies but usually is provided for up to one academic year.

Comments

Complete applications are due by February 1 for the coming academic year.

Be sure to note test requirements for application to the Graduate School. Application for GRE and TOEFL tests must be made early in autumn.

World Wide-Web access: http://www.engr.washington.edu/activities/valle/

Application Process

Requirements:

Completed Valle application
Completed UW application
Graduate School applic. fee
Official transcripts
Official TOEFL test score
Official GRE test score
Letters of recommendation
Study plan
Portfolio (Arch & Urb. Pl.)

Other Contacts

Prof. Arild Eikum
Aquateam-Norwegian Water Tech Center
P.O. Box 6326 Etterstad
0604 Oslo 6, Norway
Tel: 47 22 67 93 10
Fax: 47 22 67 20 12

Prof. Tommy Lindell
Centrum för bildanalys
Lägerhyddvägen 17
S-75237 Uppsala, Sweden
Tel: 46 18 18 34 64
Fax: 46 18 55 34 47

Note: The Seattle phone numbers below have an area prefix of 206

International Services Office
543-0840

UW Graduate Admissions
543-5929

FIUTS (Foundation for International Understanding Through Students)
543-0735

Civil Engineering Department
543-2390

Environmental Engr & Science
543-2547

Struct, Geotech Engr & Mech
543-8883

Transp, Survey, & Constr Engr
543-7331

Architecture Department
543-4180

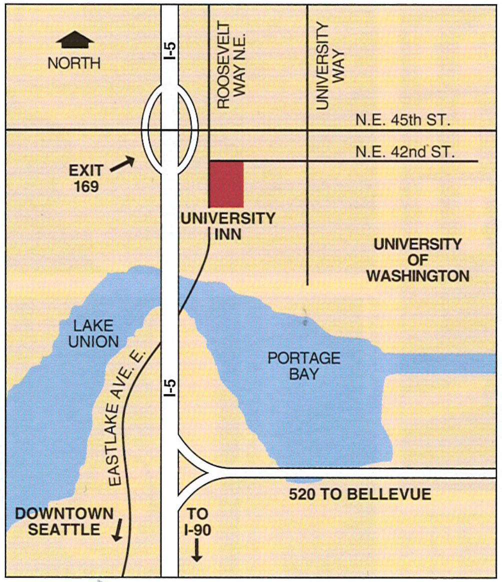
Landscape Architecture Department
543-9240

Urban Design & Planning Dept
543-4190

Vedlegg 2

University District

The renowned University of Washington, founded in 1861, has an enrollment of nearly 35,000 students. Its beautiful 694 acre campus is bordered by Seattle's colorful University District, comprised of 300 stores and specialty shops, various galleries, movie and live theaters, churches of all denominations, and scores of restaurants — from basic American to exotic ethnic. Something special is always happening here, like boat festivals, parades, summer Saturdays farmers' market, crew races, and street fairs.



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" CASE HISTORIES OF SOIL NAILING AND REINFORCED EARTH IN NORWAY"

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Technical Report Documentation Page

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16. Abstract: The long-term performance of soil nail walls has been proven after 20 years of use in Europe and the United States. The purpose of this manual is to facilitate the implementation of soil nailing into American transportation design and construction practice and to provide guidance for selecting, designing, and specifying soil nailing for those applications to which it is technically suited and economically attractive. A comprehensive review of current design and construction methods has been made and the results compiled into a guideline procedure. The intent of presenting the guideline procedure is to ensure that agencies adopting soil nail wall design and construction follow a safe, rational procedure from-site investigation through construction. This manual is practitioner oriented and includes: description of soil nailing concept and applications; summary of experimental programs and monitoring of in-service walls; recommended methods of site investigation and testing; recommended design procedures for both Service Load Design (SLD) and Load and Resistance Factor Design (LRFD); worked design examples; simplified design charts for the preliminary design of cut slope walls; wall performance monitoring recommendations; discussion on the practice and quality control of shotcrete application in soil nailing; discussion of contracting procedures and guidance on the preparation of plans and specifications; guide construction specifications and example plan details; quality control checklist for soil nail design and construction; presentation of procedures for determining the structural capacity of nail head connectors and wall facings, including demonstration calculations. This manual is intended to be used by civil engineers who are knowledgeable about soil mechanics and structural engineering fundamentals and have an understanding of the principles of soil-reinforcement technology and earthwork construction. Throughout the manual any areas of incomplete understanding of the behavior of soil nail systems are noted. A companion document titled FHWA Soil Nailing Field Inspector's Manual (FHWA-SA-93-068) is also available from NTIS.

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ENGLISH TO METRIC (SI) CONVERSION FACTORS

The primary metric (SI) units used in civil and structural engineering are:

length	-	meter (m)
mass	-	kilogram (kg)
time	-	second (s)
force	-	newton (N) or kilonewton (kN)
pressure	-	pascal (Pa = N/m^2) or kilopascal (kPa = kN/m^2)

The following are the conversion factors for units presented in this manual:

Quantity	From English Units	To Metric (SI) Units	Multiply by	For aid to Quick Mental Calculations
Mass	lb	kg	0.453 592	1 lb(mass) = 0.5kg
Force	lb	N	4.448 22	1 lb(force) = 4.5N
	kip	kN	4.448 22	1 kip(force) = 4.5kN
Force/unit length	plf	N/m	14.593 9	1 plf = 14.5N/m
	klf	kN/m	14.593 9	1 klf = 14.5kN/m
Pressure, stress, modulus of elasticity	psf	Pa	47.880 3	1 psf = 48 Pa
	ksf	kPa	47.880 3	1 ksf = 48 kPa
	psi	kPa	6.894 76	1 psi = 6.9 kPa
	ksi	Mpa	6.894 76	1 ksi = 6.9 Mpa
Length	inch	mm	25.4	1 in = 25 mm
	foot	m	0.3048	1 ft = 0.3 m
		mm	304.8	1 ft - 300 mm
Area	square inch	mm^2	645.16	1 sq in = 650 mm^2
	square foot	m^2	0.09290304	1 sq ft = 0.09 m^2
	square yard	m^2	0.83612736	1 sq yd = 0.84 m^2
Volume	cubic inch	mm^3	16386.064	1 cu in = 16,400 mm^3
	cubic foot	m^3	0.0283168	1 cu ft = 0.03 m^3
	cubic yard	m^3	0.764555	1 cu yd = 0.76 m^3

A few points to remember:

1. In a **"soft"** conversion, an English measurement is mathematically converted to its **exact** metric equivalent.
2. In a **"hard"** conversion, a new **rounded**, metric number is created that is convenient to work with and remember.
3. Use only the meter and millimeter for length (avoid centimeter).
4. The pascal (Pa) is the unit for pressure and stress (Pa = N/m^2).
5. Structural calculations should be shown in MPa or kPa.
6. A few basic comparisons worth remembering to help visualize metric dimensions are:
   - One mm is about 1/25 inch or slightly less than the thickness of a dime.
   - One m is the length of a yardstick plus about 3 inches.
   - One inch is just a fraction (1/64 inch) longer than 25 mm (1 inch = 25.4 mm).
   - Four inches are about 1/16 inch longer than 100 mm (4 inches = 101.6 mm).
   - One foot is about 3/16 inch longer than 300 mm (12 inches = 304.8 mm).

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General		Units
A	design pseudo-static seismic coefficient	(g's)
A_B	area of an individual bar	(mm^2)
A_C	area of punching shear cone base at back of facing	(mm^2)
A_E	effective tension area of concrete surrounding the flexural tension reinforcement	(mm^2)
A_{GC}	cross-sectional area of soil nail borehole	(mm^2)
A_{HS}	cross-sectional area of body of headed stud	(mm^2)
A_{LB}	area of largest bar in width S_H	(mm^2)
A_{PK}	design peak ground acceleration	(g's)
A_S	area of tension reinforcement in facing panel width 'b'	(mm^2)
$A_{S,NEG}$	area of tension reinforcement in facing panel width 'b' (negative moments)	(mm^2)
A_{SN}	cross-sectional area of strut nail borehole	(mm^2)
A_{ST}	cross-sectional area of steel in strut nail	(mm^2)
A_{TOTAL}	total area of reinforcement in width S_H	(mm^2)
A_{WIRE}	area of individual wire to be spliced	(mm^2)
B	width of base of soil nailed block	(m)
b	width of unit facing panel (equal to S_H)	(m)
B'	effective width of base of soil nailed block, accounting for eccentric loading	(m)
b_{PL}	width of bearing plate	(mm)
c_D	dimensionless cohesion	-
C_F	facing flexure pressure factor	-
C_S	facing punching shear pressure factor	-

"internal" slip modes as a pseudostatic earthquake acceleration using the definition of "internal" given in figure 4.14.

3. For slip surfaces that are primarily "external" in nature (i.e., either do not intersect the nail reinforcements or intersect them to a more limited extent), the design pseudo-static seismic coefficient A will vary depending on the permanent displacements that the retaining wall can tolerate during the design event. For example, if the wall can tolerate permanent displacements of up to $250A_{PK}$ mm (where A_{PK} is the design earthquake acceleration as a fraction of gravitational acceleration), then a design seismic coefficient equal to $0.5A_{PK}$ can be assumed (section 11, appendix A, AASHTO, LRFD 1st Edition, [29]). For other tolerable permanent displacements, the appropriate acceleration coefficient can be determined in accordance with AASHTO [29].

4. For assessment of seismic bearing stability of the reinforced soil block, a design seismic co-efficient equal to 0.5 A_{PK} is recommended.

The above design methodology is demonstrated by example in chapter 5 and appendix F.

4.8 Corrosion Protection

The long-term performance of permanent soil nails requires that they be able to withstand corrosive attack from their local environment. Characteristics defining the corrosive potential of the soil environment (i.e., ground aggressivity) are summarized in section 3.1.

4.8.1 Nail Tendon Corrosion Protection

The following constitutes FHWA recommended guidelines for nail corrosion protection on U.S. Federal-aid highway projects. For **permanent** applications, soil nail corrosion protection should consist of the following:

- In non-aggressive ground, the nail section should be resin-bonded epoxied using an electrostatic process to provide a minimum epoxy coating thickness of 0.3 mm in accordance with AASHTO M-284 [39]. The intact epoxy coating will prevent tendon corrosion by isolating the tendon from the surrounding environment. In addition, the recommended minimum thickness of coating will generally prevent normal handling and construction-induced damage. A minimum grout cover of 25 mm is recommended throughout the length of the nail. Centralizers should be placed at distances not exceeding 2.5 meters center to center, and the lowest centralizer should be placed a maximum of 0.3 meters from the bottom of the grouted drill hole. The centralizers should be made from a plastic material, be attached to the nail in a way that will not impede the free flow of grout, and be sized to position the nail tendon within approximately 25 mm of the center of the drill hole.

- In aggressive ground or for critical structures[1] (e.g., walls adjacent to lifeline high volume roadways or walls in front of bridge abutments) or where field observations have indicated corrosion of existing structures, encapsulated nails should be used. Encapsulation is generally accomplished by grouting the nail tendon inside a corrugated plastic sheath. A neat cement grout containing admixtures to control water bleed from the grout is usually employed to fill the annular space (typically 5 mm minimum) between the plastic sheath and the tendon. For this type of protection, the minimum grout cover between the sheath and the borehole wall can be reduced to 12 mm.

For **temporary** applications (of less than 36 months duration) in non-aggressive ground, the soil nail grout is considered adequate protection.

4.8.2 Nail Head Corrosion Protection

If the nail is encapsulated or is an epoxy-coated deformed bar with machine threads at the upper end, the corrosion protection is terminated to expose the bare tendon at the head of the nail in order to allow attachment of the bearing plate and nut. This area may be more susceptible to corrosion than the remainder of the nail since oxygen is more readily available.

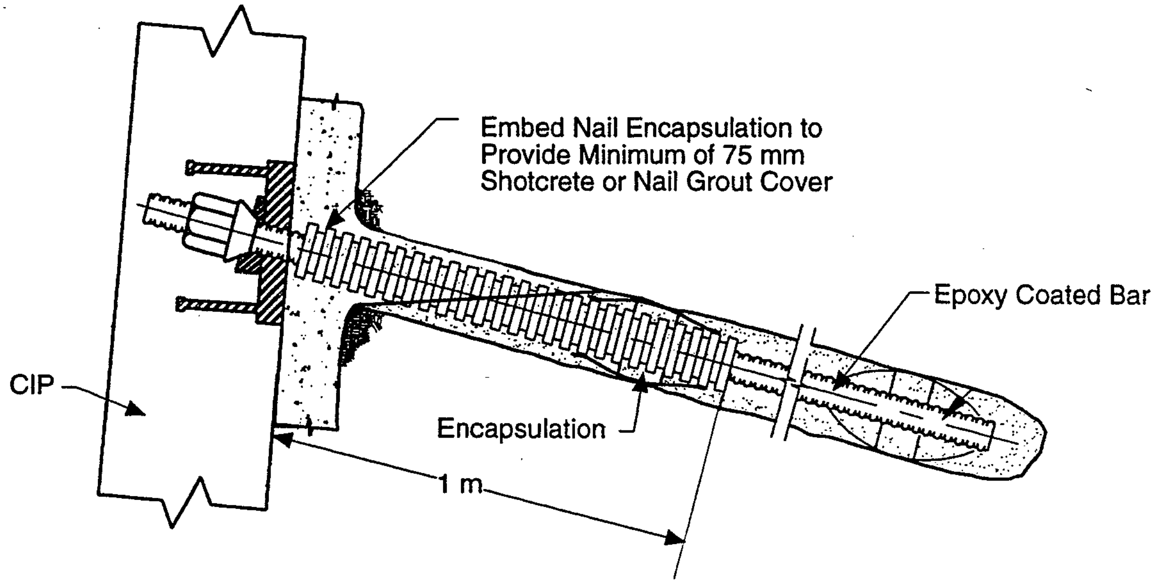
For the above type of nail tendons and corrosion protection, the following approach has been most commonly used for providing corrosion protection to the nail head. First, the bearing plate assembly is embedded in the permanent facing with the normal depths of cementitious cover to control steel corrosion. Second, the nail tendon protection (epoxy coating or encapsulation) is extended into the shotcrete construction facing to ensure a minimum depth of shotcrete/nail grout cover of 75 mm. Figure 4.15 shows examples of the types of acceptable corrosion protection systems for permanent soil nails.

When epoxy coated continuous threadbars are used (i.e., Dywidag bars), the threadbars are commonly coated full length. CALTRANS experience indicates that the use of a 0.3 mm coating thickness still allows the bearing plate nut to be threaded onto the bar over the epoxy coating. This may not be the case with other types of continuous threadbars.

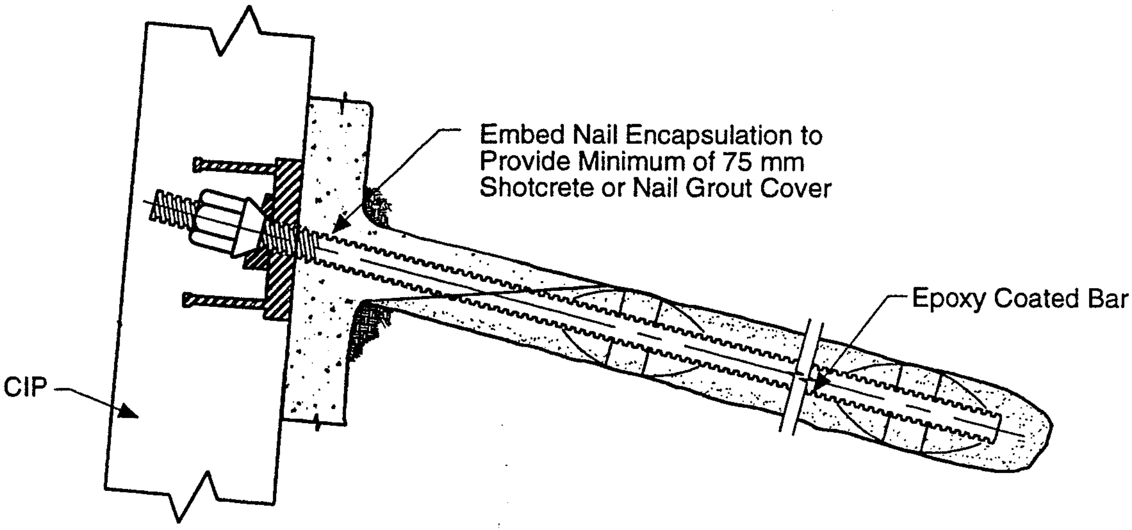
4.9 Wall Drainage

Typical soil nail wall drainage systems are discussed in section 2.3 D and include geotextile face drains, shallow PVC drain pipes and weep holes, surface interceptor/collector ditches, and surface waterproofing. Other approaches include deep horizontal drains for control of flowing water and

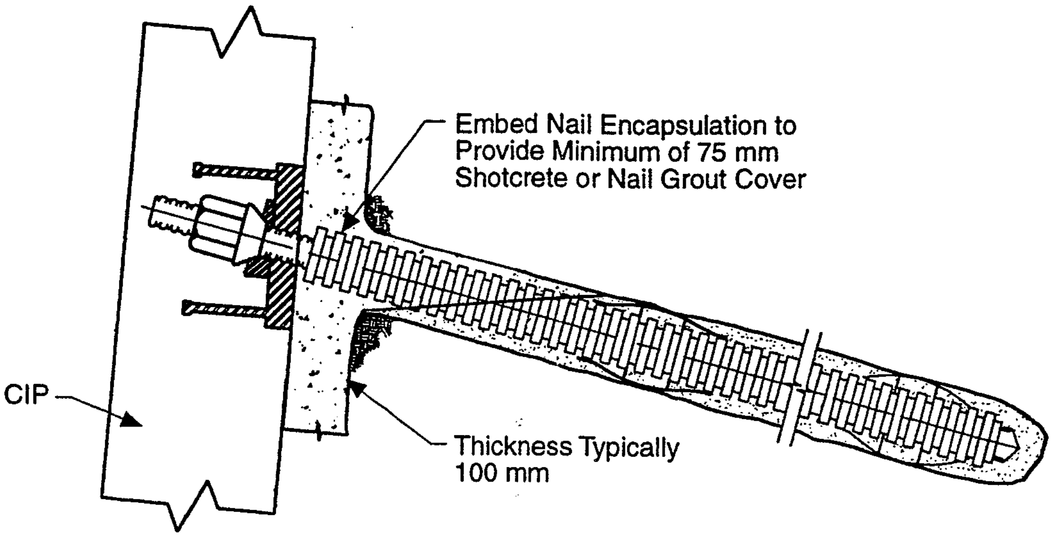
[1] *Determination of structures considered "critical" is the prerogative of individual State agencies.*



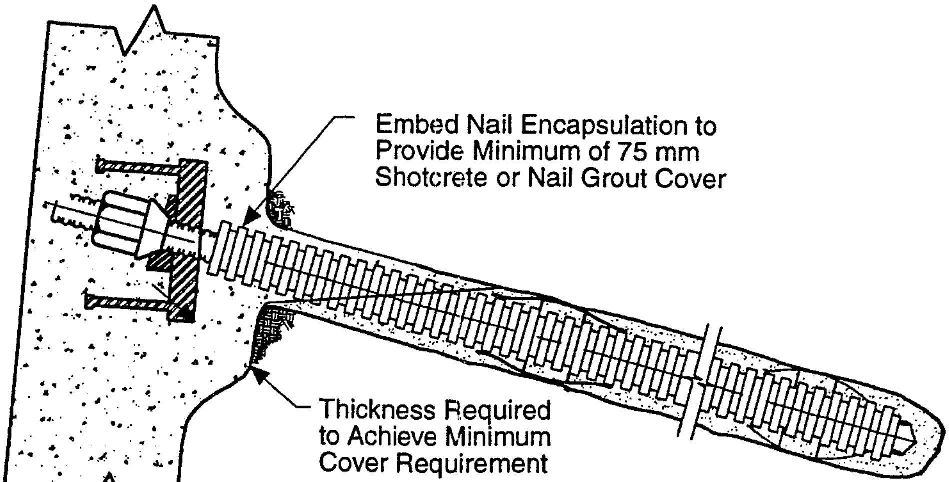
a) Epoxy Coated Soil Nail Detail (Upper 1 Meter Encapsulated) With Temporary Shotcrete Construction Facing and Permanent CIP Facing (Caltrans)



b) Epoxy Coated Nails With Machine Threads Detail, With Temporary Shotcrete Construction Facing and Permanent CIP Facing



c) Encapsulated Soil Nail With Bare Steel at the Top Detail, With Temporary Shotcrete Construction Facing and Permanent CIP Facing



d) Permanent Shotcrete Facing Detail With Encapsulated Nail

Figure 4.15 Alternative Soil Nail Corrosion Protection Details

for ground water depressurization when an unanticipated water table is encountered and use of vegetation with stepped or benched walls to inhibit infiltration and to lower soil water contents by evapotranspiration.

Typical permanent face drain configurations for geotextile drain strips discharging either into toe drains or through weep holes in the facing are shown on figure 4.16.

In blocky ground that produces a very rough and irregular excavation face, the placement of prefabricated drain strips against the excavation face is difficult and often impractical. In some cases, the prefabricated drain strips may be sandwiched between the shotcrete construction facing and the permanent CIP facing, with the drain placed over 50 to 75 mm diameter weep holes passing through the construction facing (see figure 4.16).

4.10 Special Design Considerations

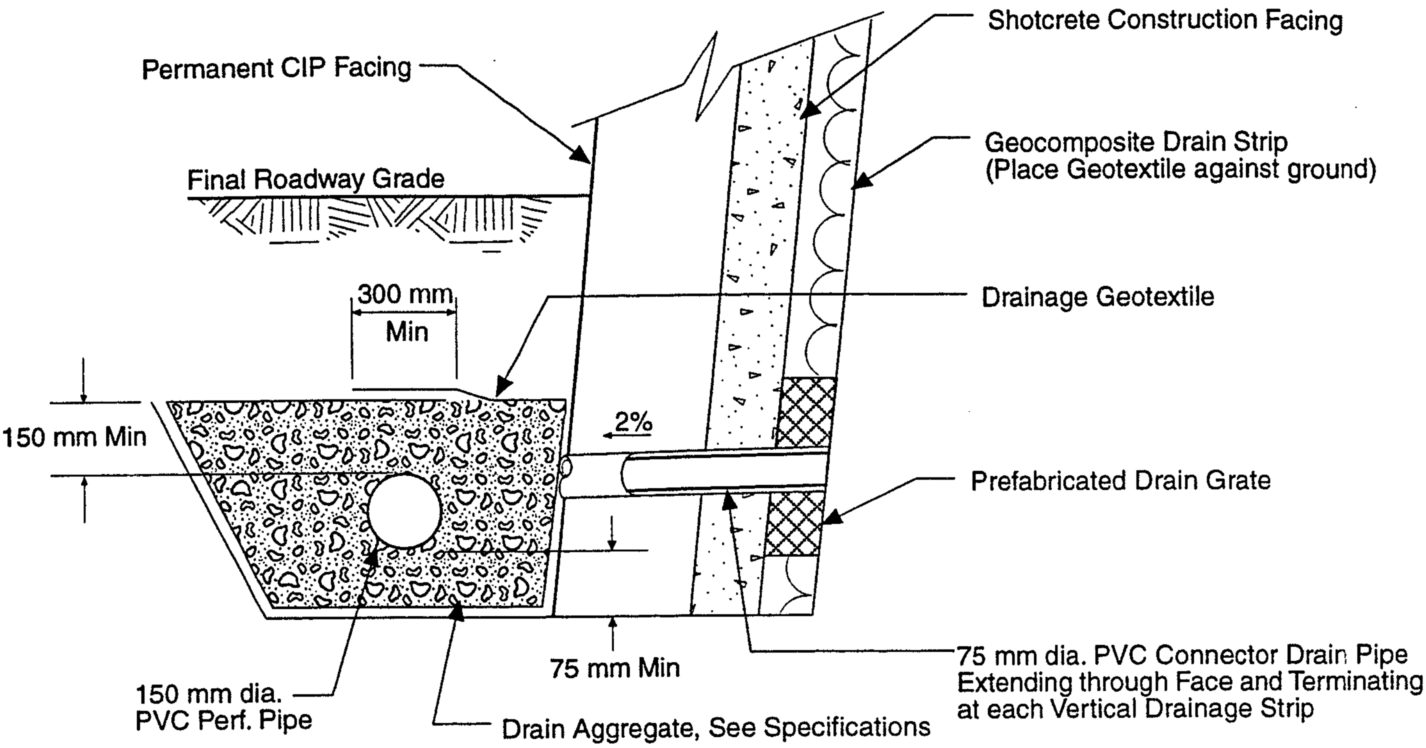
The simplest soil nail retaining wall consists of a vertical or battered planar wall, and a homogeneous soil reinforced with nails of constant length and orientation. More complex configurations are not uncommon, however, including heterogeneous ground conditions, nails of variable length and orientation, non-planar facings, and wall loadings other than those associated with the self weight of the reinforced and retained ground. These variations, discussed briefly below, can often be relatively easily incorporated into the recommended slip surface limiting equilibrium design methodology presented in section 4.7. In some applications, however, there is relatively little experience on which to base design recommendations and expert assistance should be obtained.

4.10.1 Heterogeneous Soil Profiles

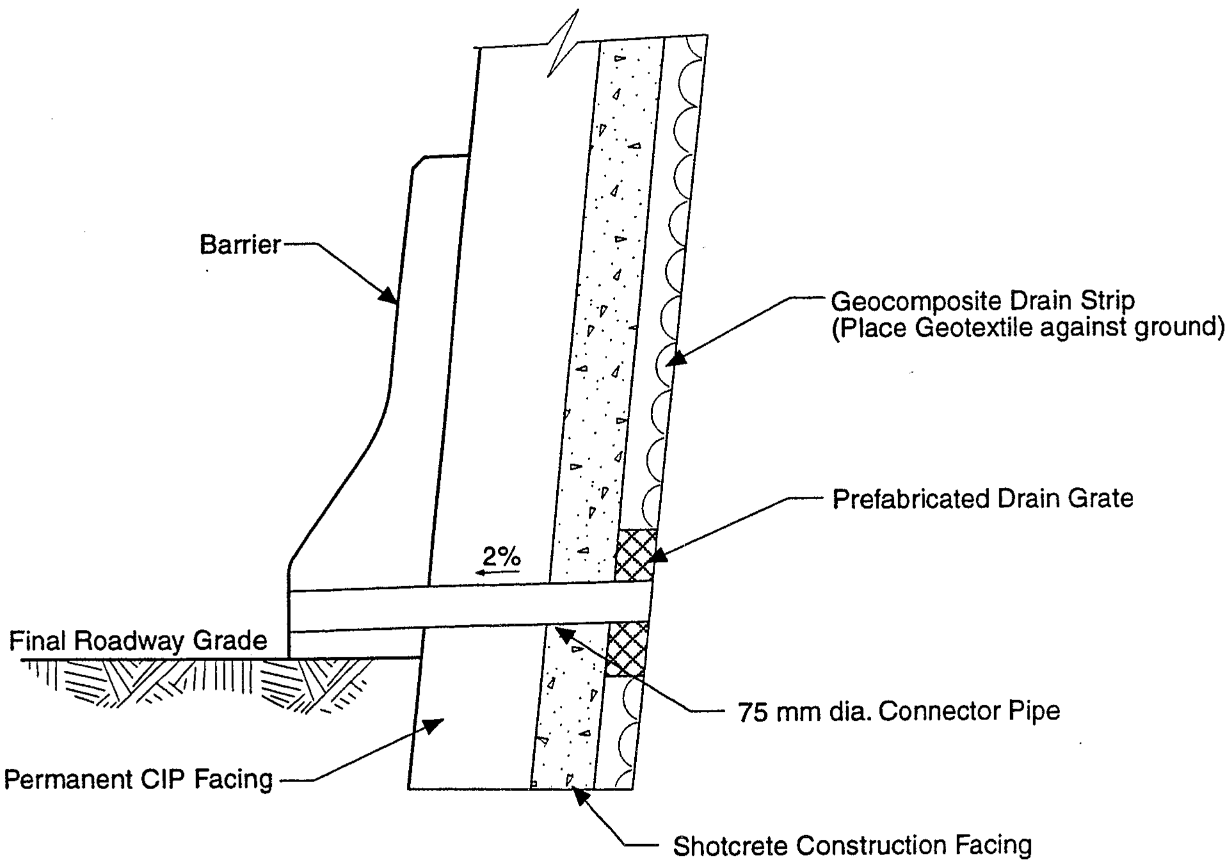
Because the soil nailing technique is concerned with the reinforcement of in situ ground rather than of controlled structural fills, it is relatively common to encounter heterogeneous conditions with respect to self weight, soil strength and nail pullout resistance. In principle, such conditions pose no particular computational difficulties for slip surface limiting equilibrium techniques. However, the following points should be considered:

- Some of the available soil nailing computer design codes are restricted to relatively simple soil profile heterogeneities, such as single uniform soil type or only horizontally layered systems.

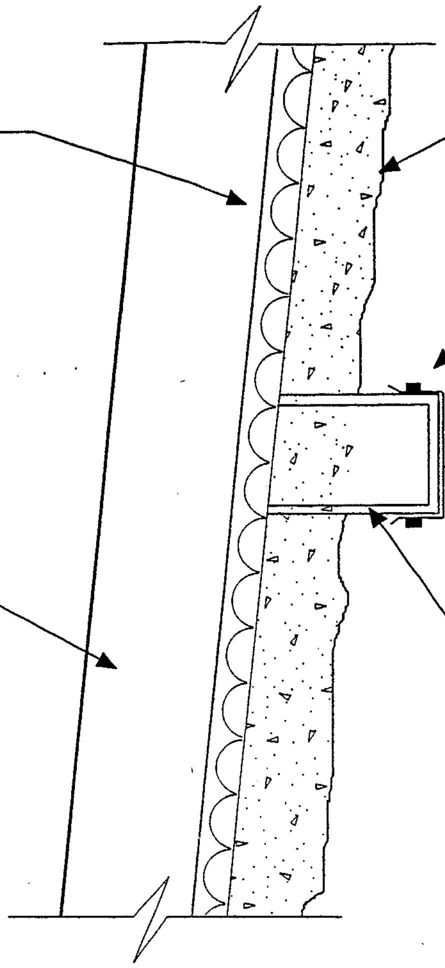
- Sensitivity studies should be conducted to examine the impacts of soil with severe heterogeneities, such as soil overlying bedrock. Good engineering judgment might require that the full rock shear strength or nail pullout resistance in the rock not be incorporated into specific zones of the design model, if it appears that this would result in an unrealistic computed factor of safety. Also, it is considered generally inappropriate to develop designs in which a small fraction of the nails are responsible for a large portion of the total nail support.



a) Typical Wall Toe Drain



(b) Typical Wall Base Drain, Traffic Barrier and PVC Connector Pipe Detail



c) Rough Excavation Face
Drain Strip Detail With Geocomposite Drain Strip Sandwiched Between Shotcrete and Permanent CIP Facing (courtesy Schnabel Foundation Co.)

Figure 4.16 Example Wall Drainage Details

- Whenever heterogeneities of any type are introduced (e.g., variable soil properties, highly non-uniform surface surcharges), the critical slip surface might not exit in the vicinity of the toe of the wall and a more complete search is generally required. An example of this condition would be a weaker soil overlying a substantially stronger soil, in which the critical slip surface might exit the wall facing in the vicinity of the weak soil-strong soil contact, rather than exiting through the wall toe (figure 4.17).

4.10.2 Surcharge Loading

Relatively uniform vertical and horizontal surcharge loadings applied to the surface of a soil nail retaining wall structure can be addressed in a similar manner to the self weight gravity load of the reinforced and retained ground. The modeling of soil surcharges should generally account for both the self weight of the soil (vertical loading) and the lateral earth pressure (horizontal loading) exerted by the fill. All potential slip surfaces must be evaluated and minimum factor of safety requirements met in accordance with the criteria presented in section 4.7. The surcharge loadings may range from relatively light (e.g., nominal live load allowances for equipment and traffic operating above the retaining wall) to very heavy loads in relation to the weight of the retained ground (e.g., surcharge corresponding to an MSE or conventional retaining structure or bridge abutment spread footing located on top of the soil nail retaining wall). The minimum facing/connection system strength requirements should also be established in accordance with the recommendations of section 4.7, taking into account the loads applied by both the ground self weight and the surface surcharges. It should be noted that there is currently little published information on measured service nail and facing loads for soil nail walls loaded with heavy surface surcharges, although a few such instrumented walls have been built on recent U.S. highway projects (e.g., I-405 Renton, WA; Portland LRT, Portland, OR).

For relatively uniform surface surcharges and homogeneous soil profiles, the critical slip surfaces giving the lowest calculated factors of safety will tend to pass near the toe of the wall and not through an intermediate point higher up in the wall.

4.10.3 Bridge Abutments

As discussed in chapter 1, one of the most useful applications of the soil nailing technique in highway construction and improvement projects is for bridge underpass widening (figure 2.3). This activity requires the removal of lateral restraint in the vicinity of the bridge foundation, by excavation of the bridge abutment retaining slope, and the almost simultaneous replacement of the removed lateral support with a soil nail retaining wall.

For soil nail walls constructed in front of existing shallow or deep foundation supported bridge abutments, the estimated top of wall and abutment movement that will be induced by the wall excavation must be able to be tolerated by the abutment and superstructure. This will be an especially important design consideration for abutments on shallow foundations, particularly where the wall face excavation would have to be made close to the foundation footing. It is emphasized that this is a critical application and should be

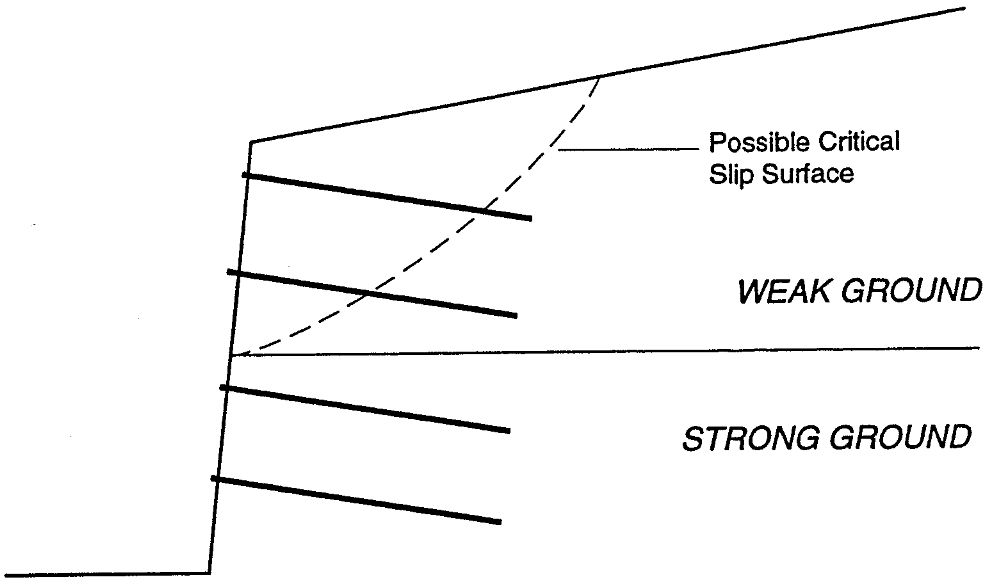


Figure 4.17 Influence of Heterogeneous Ground Conditions on Critical Slip Surface

considered only for sites where the shallow bridge footing is supported by very competent ground.

Where the bridge is supported on a deep foundation such as vertical or battered piles or piers, and the deep foundation extends well below the base of the wall such that its bearing capacity will not be significantly influenced by the slope removal and wall construction, it is recommended that the deep foundation and the soil nail retaining wall be considered as essentially independent systems. For example, the bridge vertical loads might be carried exclusively by the deep foundation and the soil nail retaining wall will be proportioned to carry the surcharge loads associated with the bridge approach fill, together with the other typical dead and live surface loads. In this respect, the design problem is no different from a typical cut slope application.

However, there are some additional issues that must be considered for such an application:

- Some fill soils may not be well suited for soil nailing. Examples include clean, loose granular soils (poor stand-up time) or fills containing numerous cobbles, boulders, rubble or other obstructions (difficult excavation and drilling).

- The presence of the piles or piers behind the future retaining wall will place restrictions on the nail layout, in particular on the nail horizontal spacings and nail-head locations.

- In addition to the vertical loads that are supported directly by the deep foundation, the bridge abutment will also be subjected to lateral earth pressure loads associated with the approach fill, as well as horizontal loads associated with longitudinal bridge temperature shrinkage and expansion. Unless the deep foundation can be demonstrated to have sufficient lateral stiffness to support these horizontal loadings with minimal horizontal displacement (e.g., battered piles), it is recommended that these horizontal loads be applied as lateral surcharge loadings in the nail wall design, together with the vertical surcharge loads associated with the approach fill. This approach is demonstrated by the bridge abutment example problem in chapter 5. In addition, any longitudinal bridge movement due to temperature shrinkage and expansion that can be transmitted into the nail wall must be assessed and judgement made as to whether it is tolerable and can be accommodated by the nail wall.

- Water flows in existing bridge drains must be controlled.

Where ground conditions are suitable and very competent (e.g., very dense soil or weathered rock with favorable geologic structure), a soil nail retaining wall can also be used to achieve bridge underpass widening where the bridge is supported by shallow foundations. Once again, the surface surcharge loading imposed by the shallow footing does not pose any particular problems from a design analysis perspective, although the following consideration must be addressed:

- The surface loading will tend to be non-uniform, with a higher bearing pressure applied over the relatively narrow width of the footing. As with other non-homogeneities, a broader range of potential slip surfaces must be considered as the critical slip surface location will tend to depend on the location of the concentrated footing loading behind the facing (figure 4.18).

Particular attention must be paid to the details of the construction process, including the temporary conditions that will exist during construction in the periods between lift excavation and nail and facing installation. It must be ensured that stability will not be compromised at any stage during the construction excavation process, and not just for the final configuration. Stability analyses should therefore be conducted for all potentially critical intermediate conditions to ensure that the foundation is not temporarily compromised by removal of lateral restraint (figure 4.18). Assessment of stability during construction is a requirement for all soil nail walls, but is of particular importance in applications that include significant surcharge loads. As noted in section 4.7, temporary construction conditions should be assessed with reduced soil strength (or global) factors of safety (SLD) or increased soil strength resistance factors (LRFD).

Some photo examples of permanent soil nail walls used in bridge end slope removal applications on U.S. highway projects in front of both deep and shallow foundation supported bridge abutments are shown in the FHWA "Soil Nailing Field Inspector's Manual" [26].

4.10.4 Stepped Structures

Aesthetic requirements may call for the use of a stepped or benched facing for a soil nail retaining wall, with horizontal setbacks between the individual wall sections (figure 4.19). These setback areas are often used for planting vegetation i.e., "greening" the wall. Where horizontal setbacks are "small" in relation to the height of the individual benches, the structure will tend to act as a single equivalent wall with a battered face. Conversely, where the horizontal setbacks are "large" (typically 1.5 times the height) in relation to the height of the individual benches, the individual benches will tend to act as totally independent walls. Since the definition of "small" and "large" in this context is dependent on the material properties, it will generally be necessary to evaluate the stability both of the overall wall and of sections of the wall comprised of single (and possibly multiple) benches and to design for the most critical case.

4.10.5 Composite Structures

Composite structures involving the use of soil nailing may take a variety of forms, together with very different construction sequences.

(a) Nails and Tiebacks

One type of composite structure is that in which nails are combined with a second method of support (typically active tieback anchors), and both supporting elements are installed as excavation proceeds from the top down. The design methodology will depend on the configuration of the support system, and particularly on the relative contribution and intended function of the nails and the tiebacks. Figure 4.20 shows two basic composite nail-tieback

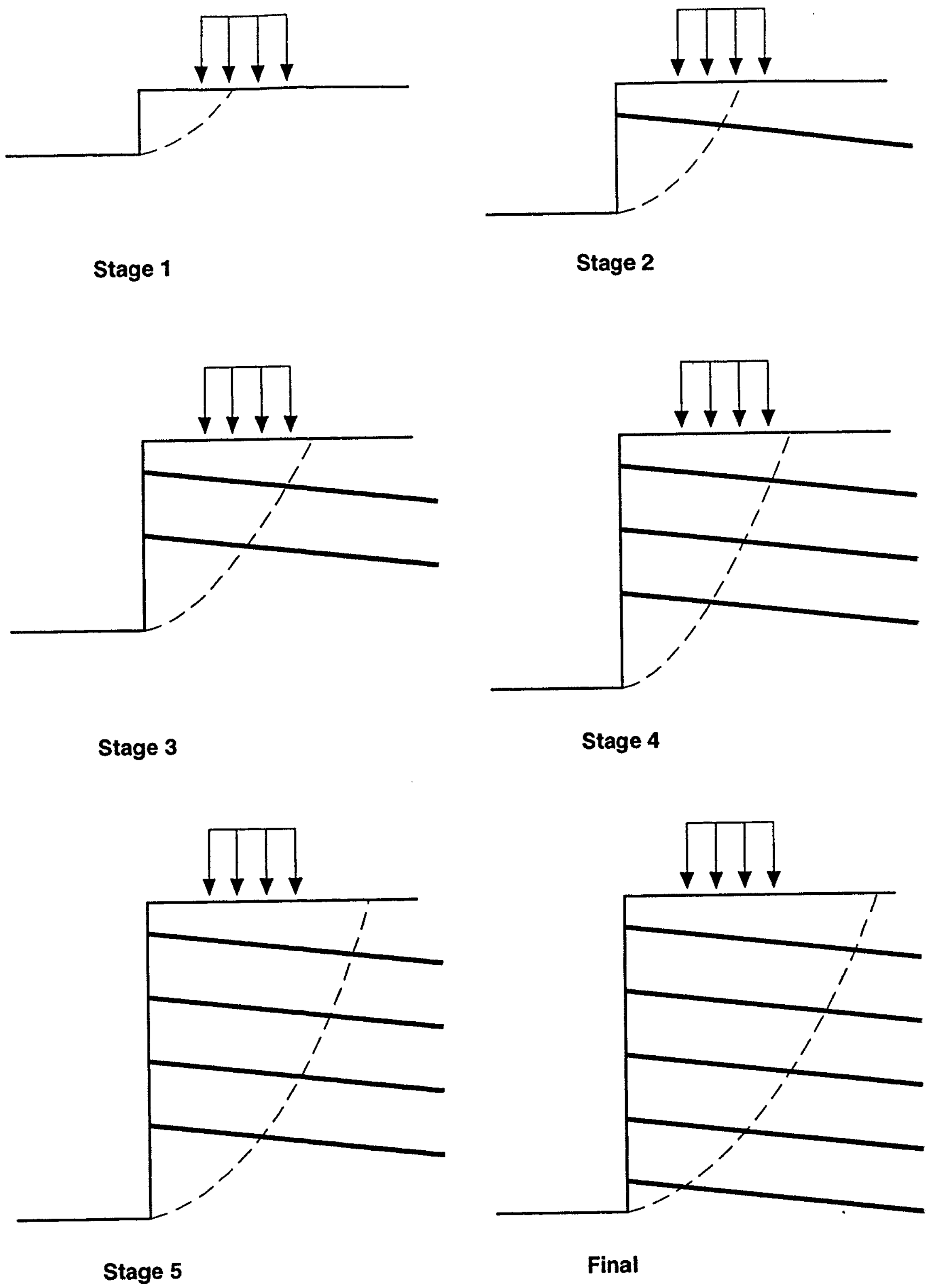


Figure 4.18 Stability Assessments for Construction Conditions

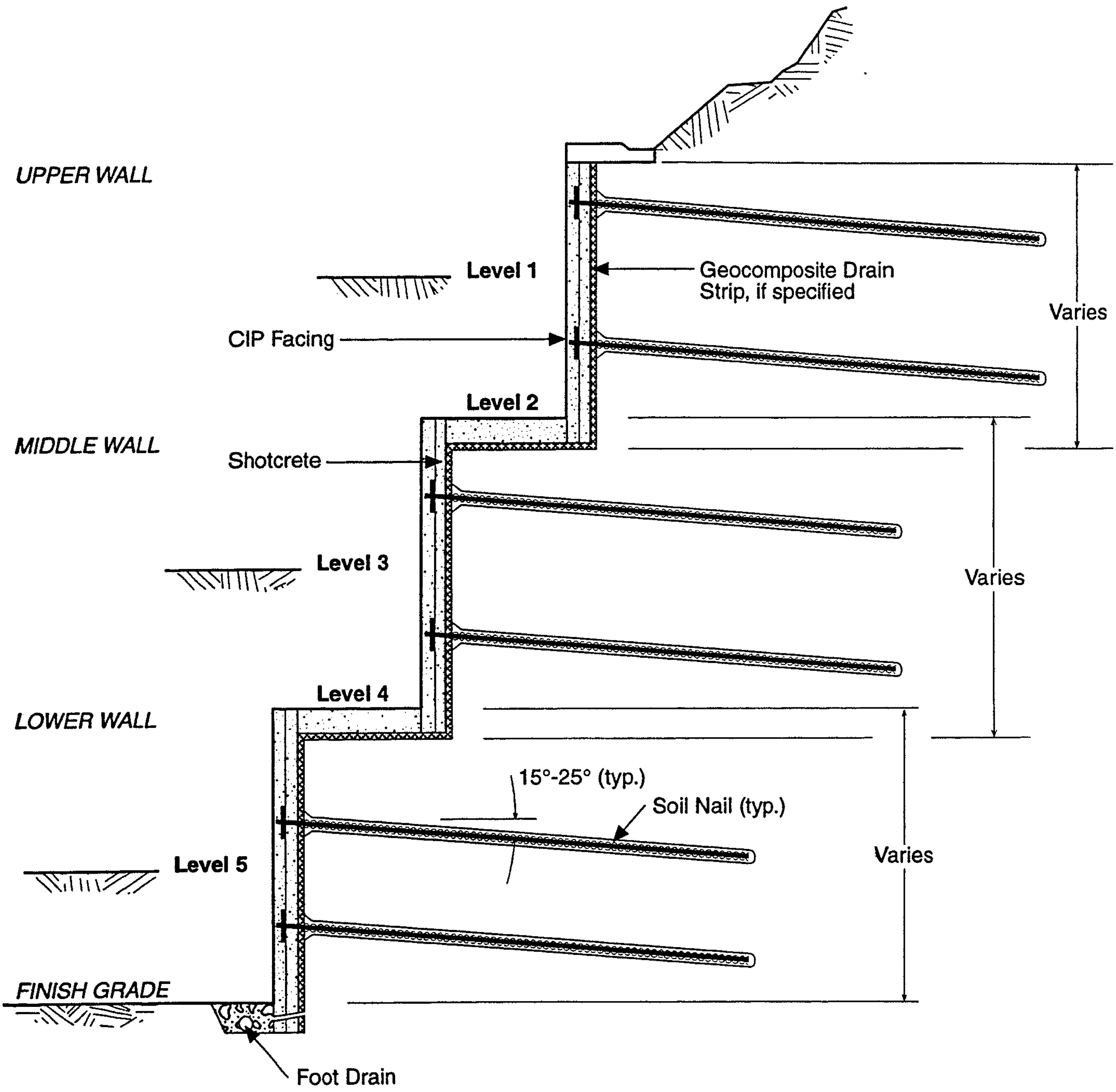
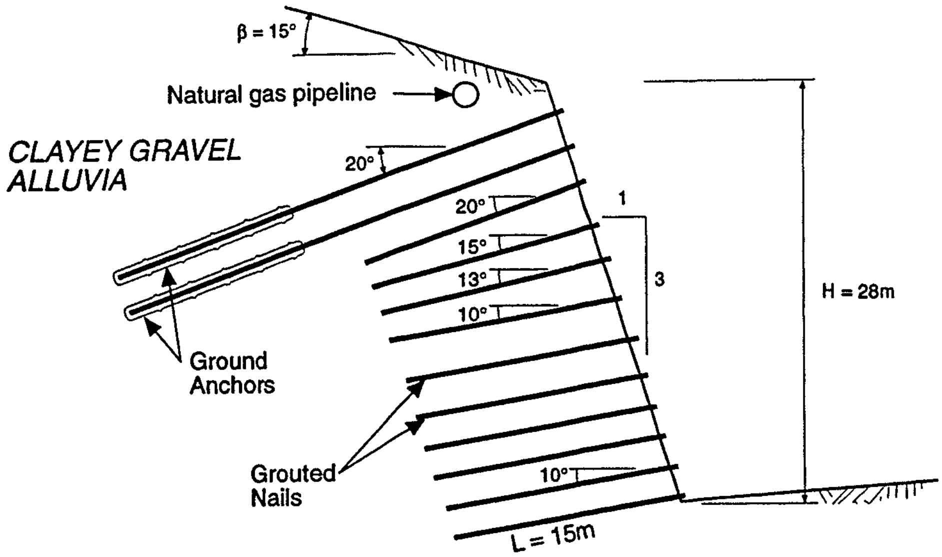
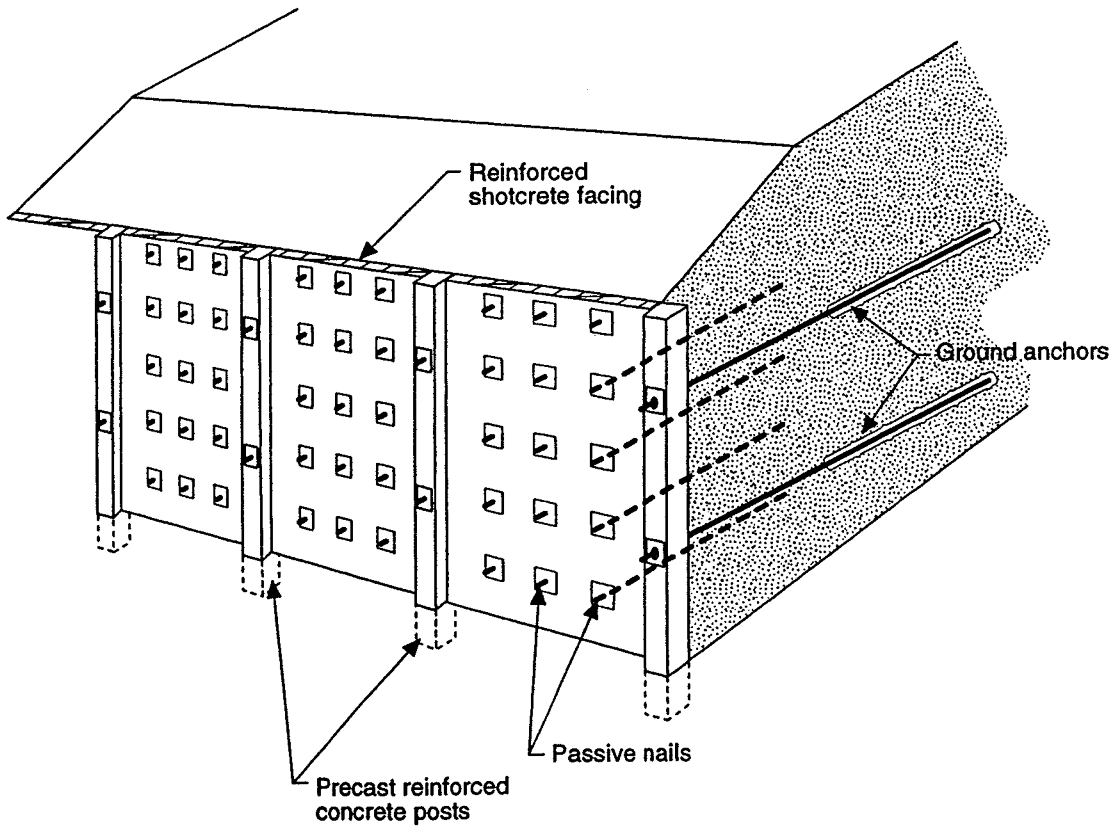


Figure 4.19 Typical Section Through Stepped Wall



(a) The Wall at the North Entrance of the Cotière Tunnel (TGV Rhône-Alpes, 1990)



(b) Nailed Berlin Wall

Figure 4.20 Composite Wall Structures

support systems that have been used and demonstrates fundamentally different support concepts requiring different design approaches.

Figure 4.20(a) shows a composite nail-tieback support system applied at the north entrance to the Cotiere Tunnel [10]. In this application, a conventional soil nail support system is complemented with tiebacks in the upper part of the wall to provide additional stability against deep-seated failures and to limit displacements in the upper wall area in order to protect a nearby critical structure. Problems with deep-seated stability can occur where significant backslopes exist and material strengths are modest. The recommended design approach for this type of application is to use the method outlined herein for the soil nail portion of the support system, but to limit the slip surfaces considered so that the deep-seated slips are not addressed (e.g., limit slip surfaces to those that intersect the ground surface at a distance of no greater than 1.5 times the proposed nail length behind the top of the wall). Nail lengths will typically not be less than are required for good soil conditions and a horizontal backslope. The soil nail reinforced zone can then be considered to act as a gravity retaining structure and the tiebacks then designed to provide the additional support required to stabilize more deep-seated slips and prevent overturning of the wall. The grouted tieback anchorages should be placed behind both the nailed zone and the most critical deep-seated slip surface affecting the whole structure.

Figure 4.20(b) shows an alternative nail-tieback support system (nailed Berlin wall), where the primary support comes from the tiebacks and short nails are used to provide more localized face stability during construction. In the application shown on figure 4.20(b), the nails also make it possible to increase the distance between the soldier piles by reducing the bending moments in the facing.

(b) Supporting Other Wall Types

A fundamentally different type of composite structure from those discussed above is an earth retaining structure such as an MSE wall or conventional retaining wall constructed on top of the soil nail wall following completion of the nailing (figure 4.21). In these applications, the nail-reinforced ground comprises the foundation for the upper structure, and it is recommended that the bearing pressures beneath both the upper retaining wall (taking account of vertical loads, horizontal loads, and overturning moments) and its retained earth be considered as surface surcharge loads for the design of the soil nail portion of the structure.

4.10.6 Structures with Variable Nail lengths

There are no design computation restrictions to considering nails of variable length, although service load monitoring indicates that wall displacements will be minimized if nail lengths are kept relatively uniform, particularly in the upper two-thirds to three-quarters of the wall height. Such a nail length distribution should therefore be the objective of the design. In the lower part of the wall it is often permissible to shorten the length of the nails since the soil-nail interaction in this region is such as to induce maximum nail loads closer to the head of the nail.

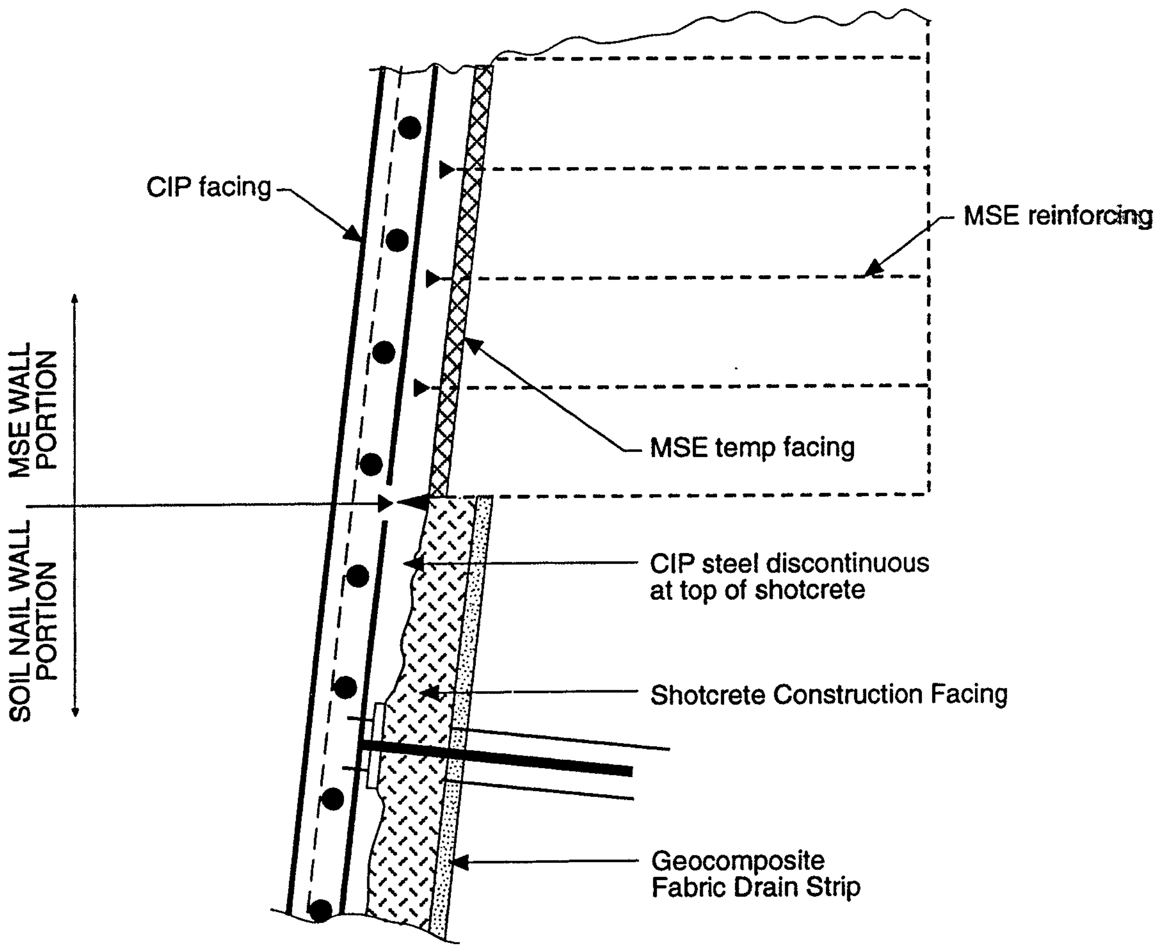


Figure 4.21 Example of Soil Nail Wall Supporting MSE Wall (Oregon DOT Portland, OR Light Rail Project)

It is often necessary to place restrictions on the length of the upper row of nails at specific locations to avoid interference with utilities, for example. The nails must be sufficiently long, however, to provide for local stability of upper wedges of the ground. Restricting the length of upper nails will limit, or virtually eliminate, their contribution to the stability of larger zones of ground associated with the overall wall height and will also reduce their ability to control surface displacements. Since the top-of-wall deformations will be reduced by having full length reinforcement within the upper part of the wall, it is desirable to minimize the use of "short" nails in the upper rows to the extent possible. However, use of short nails in the top row of soil nail walls is relatively common and has been successfully applied, particularly in urban environments, to avoid utilities.

4.10.7 Structures with Variable Nail Inclinations

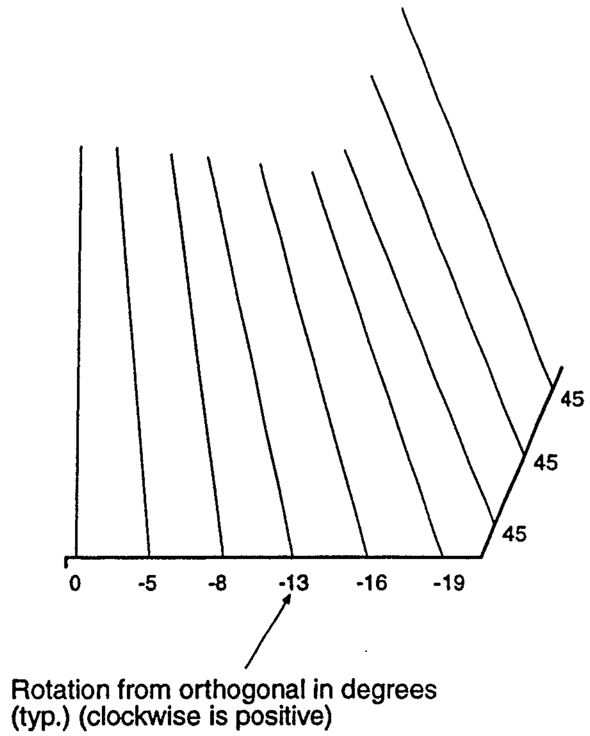
There are no design computation restrictions in the consideration of nail patterns with variable nail inclinations. From the perspectives of improving the reinforcing efficiency of the nails and also of limiting wall displacements, however, it is desirable to install the nails as close to horizontal as possible. There should, therefore, be no general incentive for installing nails more steeply than about 15 degrees below horizontal, which is the typical minimum declination required to enable grouting of the nail holes under gravity or low pressure. As discussed above for variable nail lengths, it is often necessary to steepen the inclination of the upper row of nails somewhat (e.g., to 20-25 degrees) to provide for utility clearance. When nailing under bridge decks, it is sometimes necessary to install the top row of nails at an inclination of less than 15 degrees because of limited clearance for the drill rig mast beneath the deck. Special grouting procedures will generally be required with shallow nail inclinations to ensure proper installation of the nail.

4.10.8 Structures with Variable Nail Orientations

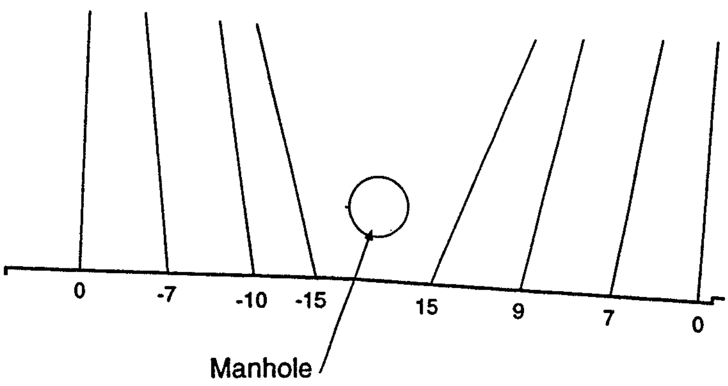
Nails will generally be installed in a vertical plane that is normal to the wall facing. For planar walls or for walls with inside curvature, this approach should be followed. For walls containing outside curvature, however, it will often be necessary to install nails in a splayed pattern that is not normal to the wall facing because of the problem of adjacent columns of nails interfering with each other. An example of this situation is shown on figure 4.22. In such instances, it is recommended that the soil nail wall design analyses be performed as though the nails were installed in vertical planes normal to the wall facing. The splayed nails actually installed should then be of sufficient length to extend to the depth normal to the facing as required by the analysis. Similarly, the component of the installed nail strength in a direction normal to the facing should equal or exceed that required from the design analysis.

4.10.9 Ground Water Seepage Forces or Water Table Close to Wall Base

As with slope stability problems, ground water and the associated seepage forces can significantly affect the stability of a soil nail structure. As discussed in section 2.2, the use of soil nailing in situations below the ground water table is not generally recommended. There may be special



Splay Nail Layout
Exterior Corner



Splay Nail Layout
Manhole Clearance

Figure 4.22 Splay Nails

circumstances where consideration of seepage forces is required, such as when an unanticipated ground water table is encountered above the wall base. Most of the problems associated with the presence of ground water and the associated seepage forces are related to constructability, and these issues must be considered by personnel experienced with such conditions. From a design perspective, however, the inclusion of seepage forces in the stability analysis poses no special difficulty and most of the computer design models currently available permit the inclusion of water seepage pressures in the analysis.

As noted in Step 8 (sections 4.7.1 and 4.7.2), a ground water table that is close to the base of the wall will tend to promote more deep-seated instabilities that pass beneath the base of the wall and exit downslope or in front of the tow of the wall. This condition must be checked for in design.

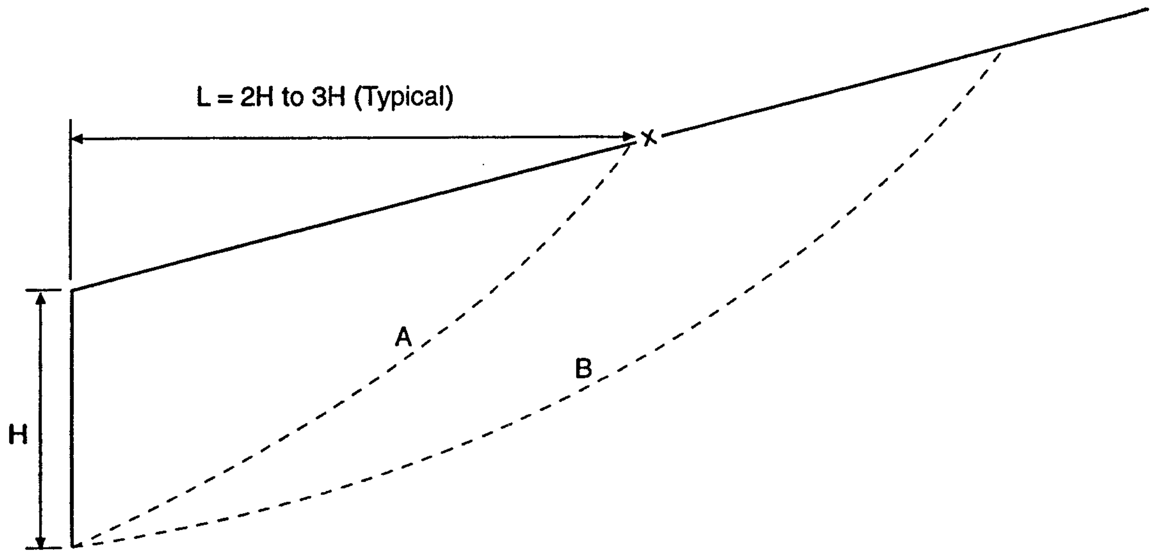
4.10.10 Infinite Slope Condition

In instances where a soil nail retaining wall is installed at the toe of a long ("infinite") slope, and that slope has a calculated factor of safety (e.g., 1.25) less than that recommended for the retaining wall itself (e.g., 1.35), then the design requirements may have to be modified. The reason: if potential slip surfaces considered in the design of the reinforced soil nail wall are unrestricted, then very large failure zones that encompass essentially the entire slope will be considered. The analysis may therefore indicate that very long, high capacity nails are required with the nails being installed in the toe region of an essentially "infinite" slope. This is generally considered to be an inappropriate use for such a retaining system. Under these circumstances, it is recommended that 1) the overall stability of the slope be independently determined and modified by other methods, if necessary, and 2) the soil nail wall be designed by limiting the scope of potential slip surfaces to the immediate area of the wall itself e.g., to within typically two to three times the height of the cut from the top of the proposed wall. This recommendation is shown graphically on figure 4.23.

4.10.11 Performance Under Seismic Loading

Recent experience has demonstrated that soil nail walls perform well under seismic loading. A number of observations of the performance of soil nail walls was made for the October 17, 1989 Magnitude 7.1 Loma Prieta earthquake in California. A post-earthquake report [4], presents field observations on eight soil nail walls in existence in the San Francisco Bay area during the earthquake. The walls, varying in height between 2.7 meters and 9.8 meters, were the subject of detailed post-earthquake visual inspections. In some cases, nails were retested after the earthquake.

None of the walls showed signs of distress even though one of them was located in Santa Cruz, an area that experienced significant seismic related damage. A 4.6-meter-high wall located on the University of California Santa Cruz campus approximately 18 km from the earthquake epicenter experienced a horizontal ground acceleration estimated at 0.47 g. Soil conditions at the site consist of a hard clayey sandy silt. Construction of this wall was completed less than three weeks before the earthquake. Prior to the earthquake, some wall footings had also been poured at the



Slip Surface Type A – Limited to slip surfaces that exit the surface at L≤ 2H to 3H behind wall. Conventional safety factors required.

Slip Surface Type B - More deep-seated slip surfaces. Required safety factors consistent with overall slope stability requirements.

Figure 4.23 "Infinite" Slope Conditions - Design Approach

bottom of the excavation immediately in front of the wall. The post-earthquake inspection revealed significant cracking of the concrete footings. This cracking was not attributed to shrinkage, since foundations constructed after the earthquake showed fewer cracks. Inspection of the soil nail wall revealed no cracking or other distress. Subsequent pullout testing of nine nails to 150 percent of their design load also indicated no loss of pullout capacity due to the seismic activity.

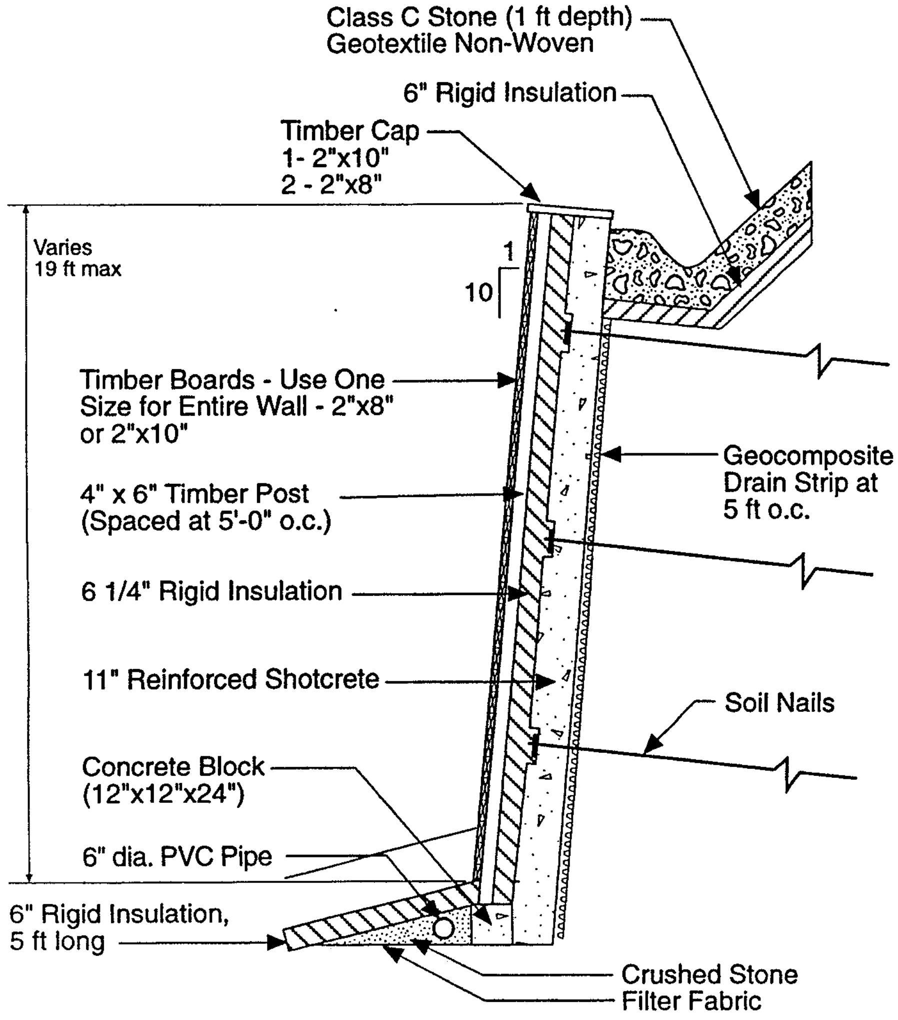
A UCLA research project co-sponsored by NSF and contractor Kulchin-Condin & Associates has also recently been performed to study in more detail the behavior of soil nail walls during earthquake loading [40]. The study included model centrifuge testing of test walls with nail length to wall height ratios ranging from 0.33 to 1.0. Tests simulating static loading indicated that for nail lengths commonly used in practice (i.e., length to height ratios greater than about 0.67), the static deformations at the top of the wall were on the order of a few tenths of one percent of the wall height. Under simulated earthquake loading, the soil nail walls performed exceedingly well. For nails of appropriate length, many cycles of shaking with peak accelerations to 0.45 g were required to induce excessive deformations of the wall. It was concluded that typical soil nail structures with grouted nails should have the capacity to resist large earthquakes, confirming the field observations from the Loma Prieta earthquake.

Both field and laboratory investigations have demonstrated the generally robust performance of soil nail walls under relatively severe earthquake shaking. For design purposes, therefore, seismic loading effects should be addressed as recommended in section 4.7. These recommendations will generally require that additional design capacity (e.g., nail length or strength) beyond that required from static design considerations may be needed only for relatively severe peak ground accelerations (i.e., greater than about 0.25 g to 0.30 g).

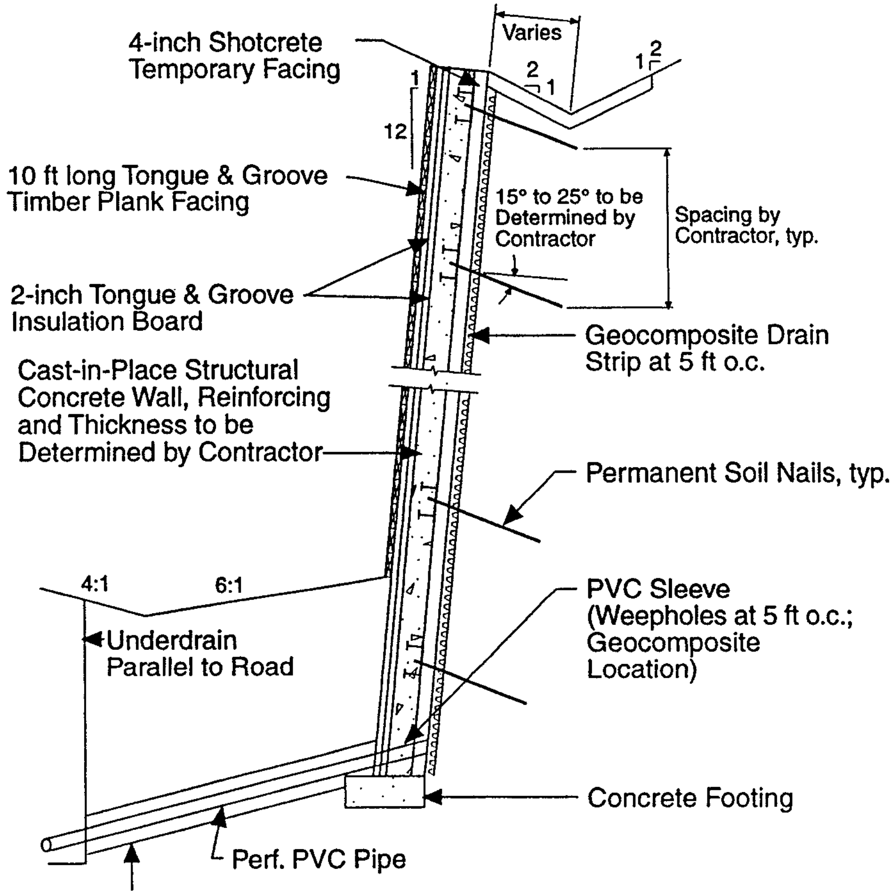
4.10.12 Frost Protection

The formation of ice lenses in the vicinity of the soil nail wall facing in frost-susceptible soils has been reported in a few cases. This has led to the development of high loads on both the facing and the head of the nail, because of the fully bonded nature of the nail and its inability to tolerate large strains in the adjacent soil without developing correspondingly high loads in the nail. This phenomenon has resulted in damage to the facing. In situations where the facing is very resistant, damage can occur to either the nail or to the connection between the nail and the facing.

The magnitude of the facing/nail loads developed will depend on the depth of frost penetration, the intensity and duration of the freeze period, and the availability of water. Increases in nail and facing loads should be anticipated in areas where frost durations are generally greater than one week and where there are frost susceptible soils near the face and in close proximity to a source of water. Frost loading effects may be eliminated or mitigated by the use of porous backfill (used on a few projects to date) or insulating material placed either between the shotcrete construction facing and the CIP or precast panel final facing, or outside the permanent concrete facing. Figure 4.24 shows proposed frost protection details using styrofoam insulation. A 1-inch thickness of styrofoam insulation board is generally considered to be equivalent to 1-foot thickness of gravel.



a) Example Soil Nail Wall with Insulation Board Frost Protection (Courtesy New Hampshire DOT)



b) Example Design/Build Soil Nail Wall With Insulation Board Frost Protection (Courtesy Maine DOT) Experimental Features Project

Figure 4.24 Frost Protection

Styrofoam insulation board has been used to insulate some permanent tieback wall facings (personal communication from D. Weatherby, Schnabel Foundation Co.). At the time of final preparation of this manual, the Maine and New Hampshire DOT Details shown on figure 4.24 are the first known soil nail walls in U.S. using insulation board for facing frost protection. Use should be considered experimental at this time.

4.10.13 Expansive Soils

At present, there are no established design procedures for soil nail walls in highly expansive soils, and this is generally considered a non-application for permanent walls. However, soil nail walls may have application in soils that are expansive in nature, particularly for temporary shoring applications, provided measures are taken to inhibit significant changes in moisture content of the soils during the life of the structure. Under such conditions, a conventional design approach such as recommended herein can be adopted.

4.10.14 Residual Soils

Residual soils will often exhibit specific slip surfaces, defined by relict structure, with shear strength characteristics that are significantly lower than those that apply to the ground mass in general. For example, two joint sets may combine to form potentially unstable wedges dipping out of the face. Provided it can be demonstrated that the relict structure will exhibit sufficient stand-up time to enable safe and economic construction of a soil nail wall, the design procedure is essentially identical to that previously discussed. However, since all potential failure modes must be considered, the analyses must address both general or non structurally controlled slip surfaces in association with the strength of the ground mass, together with specific structurally controlled slip surfaces in association with the strength characteristics of the relict joint surfaces themselves. The soil nail reinforcement must then be configured to support the most critical of these two conditions.

4.10.15 Structures with Externally Loaded Wall Facings

The nails and permanent facing of a soil nail retaining wall may be required to support external loads, and associated shear forces and bending moments, applied directly to the facing, in addition to those developed by the nail-ground-facing interactions during construction of the wall. Most commonly, these external loads will be applied at the top of the wall facing and may vary from relatively light highway appurtenance loads (e.g., roadway lighting supports) to much more significant loads associated, for example, with the integration of a relatively large cantilever retaining structure on top of the wall. An example of the latter type of external load application is shown in figure 4.25. For relatively light loading conditions, the external loads may be used to define statically-determinate additional shear forces and bending moments in the cantilevered section of the wall above the first row of nails, together with additional loads in the top row of nails themselves. For more significant loads, it may be necessary to perform a full soil-structure

Step 3

$T_D = \Phi_N T_{NN} / (\Gamma_W \gamma H S_V S_H)$
$T_{NN} = \Gamma_W \gamma H S_V S_H T_D / \Phi_N$
$T_{NN} = (1.35)(18.0\ kN/m^3)(9.50\ m)(1.50\ m)(1.50\ m)(0.23)/(0.90)$
$T_{NN} = 133\ kN$ (Required nominal nail strength.)

Step 4

$Q_D = \Phi_Q Q_U / (\Gamma_W \gamma S_V S_H) = (0.70)(60.0\ kN/m)/[(1.35)(18\ kN/m3)(1.50\ m)(1.50\ m)]$
$Q_D = 0.77$

$T_D/Q_D = 0.23/0.77 = 0.30$

From Chart C (figure 5.33C), $L/H = 0.88$

$L = 0.88(9.50\ m) = 8.4\ m$

In summary, the design charts indicate a required bar yield strength of about 133 kN (use No. 25, Grade 420 bars, although No. 22, Grade 420 bars could also be used) and a nail length of approximately 8.4 m. This nail length could be slightly conservative since the effective backslope angle of the design section shown on figure 5.7 may be something less than 20° (i.e., the design charts are prepared for constant backslope angles only, whereas the design example backslope angle is variable). For comparison purposes, a backslope angle of 10° would indicate a required nail length of about 7.3 meters.

CHAPTER 6. SOIL NAIL WALL PERFORMANCE MONITORING

6.1 Introduction

Although several hundred soil nail structures have been constructed worldwide, only a limited number have been instrumented to provide performance data in support of design procedures. Confidence in the use of soil nail shoring structures and improvements in design will be enhanced by proven performance of such systems. To this end, it is important to monitor performance behavior of future soil nail structures. This chapter includes details necessary to plan and implement both limited and comprehensive monitoring programs for soil nail systems. Recommendations for appropriate instrumentation are included.

Safe and economic soil nail walls can currently be designed, yet the behavior of such walls is still being studied. It is considered that some current design techniques may incorporate considerable conservatism. Additional performance data are needed in order to refine the design and construction methodologies.

The United States Department of Transportation, Federal Highway Administration (FHWA) has published a document titled *Reinforced Soil Structures, Volume I., Design and Construction Guidelines*, FHWA-RD-89-043 [44]. Chapter 8 of this guideline, "Monitoring of Reinforced Soil Structures", provides a comprehensive discussion and details of appropriate instrumentation schemes, equipment requirements, etc. for providing soil nail wall performance monitoring. This chapter is recommended reading for all soil nail design engineers, inspectors, and specialty contractors.

On U.S. Federal-aid highway projects, it is recommended that the initial permanent soil nail wall constructed in each State and any initial critical or unusual installations (e.g., walls greater than 10 m high, widening under existing bridges, walls with high external surcharge loading, etc.) be designated an "Experimental Features Project" and have performance monitoring instrumentation installed. A major advantage of the Experimental Features designation is that it allows construction funds to be used to pay for the performance monitoring instrumentation and evaluation including data interpretation, plotting and report preparation. Performance monitoring instrumentation for such walls should preferably include slope inclinometers and top of wall survey points to measure wall movements during and after construction and load cells and strain gages installed on selected production nails to measure nail loads at the wall face and along the nail length. By strain gaging individual nails, the development and distribution of the nail forces may be measured to provide vital feedback to designers. Load cells at the nail head also provide data on facing loads. Monitoring for a period of at least 2 years after construction is recommended because the instrumentation monitors service behavior, i.e., structural deformation and stress development in the nails and wall facing as a function of both load and time and environmental changes such as winter freeze-thaw cycles.

6.2 Soil Nail Wall Performance Monitoring Methods

Monitoring during wall construction can be of limited nature with the intention of obtaining data on the overall wall performance. As a minimum, observations and monitoring should typically include:

- Face horizontal movements using surface markers on the face and surveying methods and inclinometer casings installed a short distance (typically 1 m) behind the facing.

- Vertical and horizontal movements of the top of wall facing and the ground surface behind the shotcrete facing, using optical surveying methods.

- Ground cracks and other signs of disturbance in the ground surface behind the top of wall, by daily visual inspection during construction and, if necessary, crack gages.

- Local movements and or deterioration of the facing using visual inspections and instruments such as crack gages.

- Drainage behavior of the structure, especially if groundwater was observed during construction. Drainage can be monitored visually by observing outflow points or through standpipe piezometers installed behind the facing.

Alternatively, soil nail wall performance monitoring can be more comprehensive and continued over a longer time period for one or more of the following purposes:

- Confirming design stress levels and monitoring safety during construction.

- Allowing construction procedures to be modified for safety or economy.

- Controlling construction rates.

- Enhancing knowledge of the behavior of soil nail structures to provide a base reference for future designs with the possibility of improving design procedures and/or reducing costs.

- Providing insight into seismic performance based on long-term performance monitoring through future earthquake events.

A more comprehensive monitoring plan might include the following:

- Strain gage monitoring along the length of the nail to determine the magnitude and location of the maximum nail load. Ideally, strain gages are attached to the nail tendon in pairs, and are mounted top and bottom at a 1.5m spacing circumferentially 180° apart to address bending effects. In either case the end of the bar should be inscribed so that the final orientation of the strain gage can be verified.

- Load cells to measure loads at the head of the nail. Higher quality nail load data near the head of the nail can generally be obtained by load cells rather than by strain gages attached to the nail. The nail section immediately behind the facing is sometimes subjected to bending due to the weight of the shotcrete facing.

- Inclinometers installed from the ground surface at various horizontal distances up to one times the wall height behind the wall facing, to measure horizontal movements within the overall structure. Sometimes it might be preferable to make use of horizontal single- or multi-point borehole extensometers, particularly if access for the installation of inclinometers is difficult or if access is available for extensometer installation prior to the installation of the nails, e.g., through an existing bridge abutment or pile cap.

A typical instrumentation layout for a comprehensive monitoring plan is shown in figure 6.1.

6.3 Soil Nail Wall Performance Monitoring Plan

A well defined, systematic plan should be developed for all monitoring programs, whether limited or comprehensive. The first step is to define the purpose of the measurements. Every instrument on a project should be selected and placed to assist in answering a specific question. If there is no question, there should be no instrumentation. Both the questions that need to be answered and the clear purpose of the instrumentation in answering those questions should be clearly established. More detailed discussions of appropriate instrumentation schemes, equipment requirements, etc., are contained in FHWA RD-89-043 [44], chapter 8.

The following list (based on Dunnicliff, [45]) provides the key steps that should be followed in developing a systematic approach to planning monitoring programs for soil nail walls using geotechnical instrumentation. All steps should be followed and, if possible, completed before instrumentation work commences in the field.

- Define purpose of the monitoring program.

- Define the project conditions.

- Predict mechanisms that control behavior.

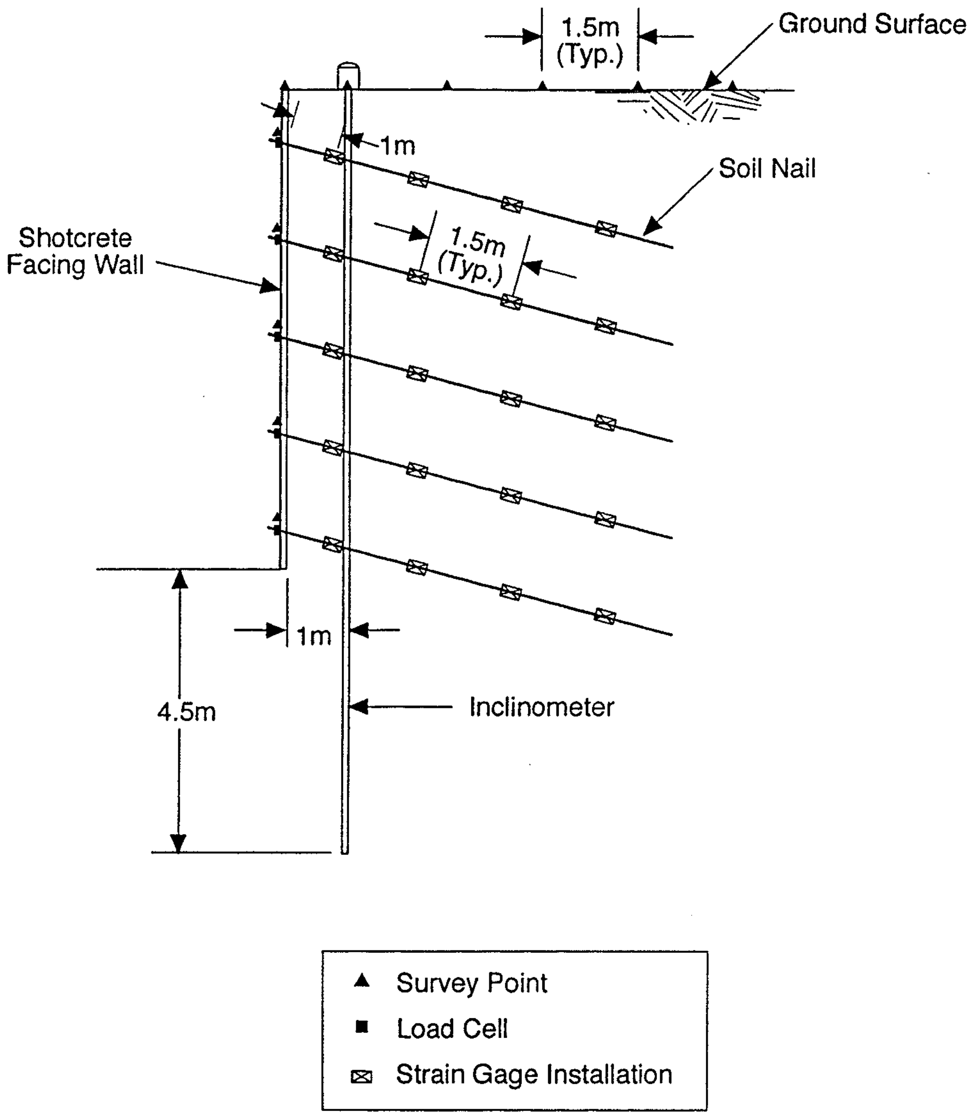
- Select the parameters to be monitored.

- Predict magnitudes of change of measured parameters.

- Assign monitoring tasks for design, construction, and operation phases.

- Select instruments, based on reliability and simplicity.

- Select instrument locations.



Note:
Number and location of instruments may be adjusted to suit field conditions.

Figure 6.1 Typical Instrumentation

- Plan recording of factors that may influence measured data.
- Establish procedures for ensuring reading correctness.
- Plan installation.
- Plan regular calibration and maintenance.
- Plan data collection, processing, presentation, interpretation, reporting, and implementation.
- Write instrument procurement and installation specifications.
- Prepare budget.
- Write contractual arrangements for field instrumentation services.

6.4 Parameters To Be Monitored

As part of developing the monitoring plan, the Engineer must select the parameters to be monitored. The most significant parameters should be identified, with care taken to identify secondary parameters that should be measured if they could influence the primary parameters. **The most significant measurement of overall performance of the soil nail wall system is the deformation of the wall or slope during and after construction.** Slope inclinometers at various distances back from the face provide the most comprehensive data on ground deformations.

The following list provides the important parameters that should be considered during development of a systematic approach to planning soil nail wall performance monitoring programs using geotechnical instrumentation:

- Vertical and horizontal movements of the wall face.
- Vertical and horizontal movements of the surface of the overall structure.
- Local movements or deterioration of the facing elements.
- Drainage behavior of the ground.
- Performance of any structure supported by the reinforced ground, such as roadways, bridge abutments or footings, slopes above the wall, etc.
- Loads in the nails, with special attention to the magnitude and location of the maximum load.

- Load distribution in the nails due to surcharge loads.

- Load change in the nails as a function of time.

- Nail loads at the wall face.

- Temperature (may cause real changes in other parameters and also affect instrument readings).

- Rainfall (often a cause of real changes in other parameters).

6.5 Soil Nail Wall Performance Monitoring Instruments

Soil nail wall performance monitoring instruments should be selected based on the parameters to be measured, the instrument's reliability and simplicity, and the instrument's compatibility with the readout devices specified for the project or already the property of the owner. Other factors to be considered include the influence of the instrument's installation on construction (e.g., access and time required) and the skills of the personnel who will read the instruments. A brief discussion of the various types of monitoring instruments typically employed for assessing soil nail wall performance is provided below.

6.5.1 Slope Inclinometers

The most significant measurement of overall performance of the wall system is the deformation of the soil nail wall during and after construction. Slope inclinometers, preferably installed at about one meter behind the soil nail wall face, provide the most comprehensive data on wall deformations.

Inclinometers are a well-established technology and are commercially available from several manufacturers. The measuring system typically consists of a portable probe that measures its own orientation relative to vertical. The probe is mounted on wheels and is raised or lowered within a grooved casing installed vertically in the ground. Readings are recorded by hand or on data loggers.

Detailed specifications for inclinometer selection and installation are presented in appendix D - Guide Specification for Soil Nail Wall Instrumentation. The following general guidelines should be followed for all installations:

- The bottom of the grooved casing should be installed in a stable portion of the ground mass, typically several meters beneath the lowest expected zone of movement.

- The casing should be aligned so that the grooves, which are typically at 90-degree intervals, are parallel and perpendicular to the wall face.

- The borehole containing the casing should be completely backfilled so that no voids are present around the casing. This is typically done with a grout mixture tremmied from the bottom of the inclinometer borehole. The grout mixture should be relatively weak (e.g., a lean cement) so that it does not provide any structural reinforcement of the surrounding ground.

- The top of the casing should be in firm contact with the surrounding soil and should be adequately protected from damage.

Details of a typical inclinometer installation are shown in appendix D.

6.5.2 Survey Points

Soil nail wall face deformation can be measured directly by optical surveying methods or with electronic distance measuring (EDM) equipment. Also, ground movements behind the soil nail wall can be assessed by monitoring an array or pattern of ground surface points established behind the wall face and extending for a horizontal distance at least equal to the wall height. In addition, reflector prisms attached to selected nails permit electronic deformation measurements of discreet points on the soil nail wall face. Frequent monitoring of the ground during the progress of construction allows the actual performance to be checked against the design assumptions. It also provides a real-time record of performance, thereby allowing modification of the construction procedure in response to changed conditions. This can be particularly useful if wall deformations become significant because poorer ground than originally anticipated is encountered or the contractor uses inappropriate construction methods.

The survey system should be capable of measuring horizontal and vertical displacements to an accuracy of 3 mm or better.

6.5.3 Soil Nail Strain Gages

Soil nails instrumented with strain gages allow assessment of the soil nail load distribution as the excavation progresses and following completion of the soil nail wall installation. By strain gaging individual nails in the laboratory and during field tests, the development and distribution of the nail forces may be measured.

Detailed specifications for strain gage selection and installation are presented in the Appendix D Guide Specification. The following general guidelines should be followed for all installations:

- Weldable vibrating wire strain gages are recommended for ruggedness and low susceptibility to electrical signal degradation.

- Temperature sensors should be included at each strain gage location to allow temperature correction. These are particularly important near the face. (Some gages contain integral temperature sensors.)

- At each measuring location on the nail, gages should be installed in pairs 180° apart in a vertical plane (top and bottom) to evaluate bending and provide an average strain reading.

- The surface of the nail should be carefully prepared and strain gage installation performed in accordance with the manufacturer's recommendations.

- The completed strain gage installation should be covered with a soft, waterproof material to ensure that (1) moisture does not penetrate the gage, and (2) the gage is mechanically decoupled from the surrounding grout.

- Preferably, some type of mechanical assembly should be installed at each gage location to break the grout column and ensure that all load is transferred to the nail bar at this point. This approach will eliminate data interpretation problems associated with grout/nail interaction. Because the grout has some tensile strength, it will carry a portion of the total load. This load will depend to a large degree on the in-place deformational characteristics of the grout and the interaction between the grout and borehole wall, both of which are difficult to evaluate. Thus, while strain measurements in the grout and nail are readily achievable, conversion of these measurements into nail loads is difficult to achieve with accuracy. An alternative approach for obtaining accurate measurements of nail load is to introduce a break in the grout at the location of the strain gage, thereby forcing the nail to carry the entire load at this location. Examples of potential approaches include a disk coated with a release agent, two disks butted together, or a soft zone around the gages formed with foam. See further discussion in the Appendix D Guide Specification commentary 3.3a.

- Signal cables should be protected by installation in conduit, from construction damage and vandalism, for example.

- In some areas of the country, lightning protection may be required.

Details of a recommended strain gage installation are shown in the Appendix D Guide Specification.

6.5.4 Load Cells at the Nail Head

Load cells installed at the soil nail head are used to provide reliable information on the actual loads that are developed at the facing.

Specifications for load cell selection and installation are presented in appendix D. The following general guidelines should be followed for all installations:

- Unless the load cells are temperature-compensated, temperature sensors should be included with each cell to allow temperature correction.

- The axis of the load cell should be aligned with the axis of the soil nail, to prevent eccentric, lateral, or other types of non-uniform loading. Hardened steel spherical bearings are preferred. Bearings and other support hardware should be well-lubricated prior to installation.

- The bearing surface under the load cell should be firm and unyielding. This will prevent apparent loss of load that could otherwise result from creep underneath the cell. High-early strength cement mortar and steel bearing plates (12-mm-thick minimum) are typically used.

- Load cells should be protected from construction damage and vandalism. Signal cables should be placed in a conduit for protection.

- In some areas of the country, lightning protection may be required.

Details of a typical load cell installation are shown in the Appendix D Guide Specification.