Muldkeha projekteerimine (sh. arvutused) ja tüüplahenduste väljatöötamine geosünteetika kvaliteedikontrolli arendamine

Arendustöö

Lõpparuanne

TRANSPORDIAMET 2021

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3. Geosünteetide kasutamine tee konstruktsioonis.
4. Lühikokkuvõte. FHWA NHI-05-037 Geotechnical Aspects of Pavements (2006).
5. Lühikokkuvõte. Design Guidance for Road Pavement Foundation (2009).
6. Lühikokkuvõte. Geotechnical Technical Guidance Manual (2007). U-S Department of Transportation. Federal Highway Administration.
7. Erinevate riikide normide lühikokkuvõte.
8. Artikkel avaldamiseks „Teelehes“.
9. Vastused Lisade 1, 2 ja 3 osas tellija detsembris 2021 edastatud Transpordiameti taristu haldamise teenistuse taristu haldamise osakonna küsimustele.

1 Sissejuhatus

Arendustöö peamiseks eesmärgiks oli analüüsida ja leida toimivad lahendused tee muldkeha ja veerežiimi projekteerimisel, ehitamisel ja järelevalvel, mis arvestavad võimaluse piires Eesti looduslikke ja Transpordiameti ning projekteerijate, omanikujärelevalve ja ehitajate ressursse nii, et oleks arvestatud tee konstruktsiooni projekteerimise terviklikkus, arvestades Euroopa ja maailma geotehnika (sh geosünteetika) toimuvaid arenguid ning kõike seda nii, et oleks arvestatud Eestis kohalduvaid nõudeid ja Eestis toimuvaid muudatusi teedeehitusega seonduvates regulatsioonides.

Transpordiamet on suurendanud oma nõuetes katendi kavandatavat eluiga, seega on vaja nii tee konstruktsiooni kui muldkeha ja sellega seonduva pikemat kvaliteetset vastupidavust. Tagamaks elutsükli kulude vähenemist koos tee konstruktsiooni eluea kasvuga tuleb senisest rohkem arvestada veerežiimi mõjusid tee konstruktsioonis ehitusmaterjalide ja pinnastele, nende ajas püsivusele ning looduses esinevaid aluspinna tugevusi (sh selle tugevdamine).

Tulenevalt vajalikust kitsamast spetsialiseerumisest teedeehituses aina enam rakendust leidvatest keerulisematest konstruktiivsetest lahendustest Eesti riigi piiratud inimressursi arvestades (Transpordiameti tehnilise koosseisu vähenemine, kus kitsama eriala, eriti geotehnika spetsialistide palkamine vahetu projekteerimise ja ehitusprotsessi tellimisse ja kontrollimisse on raskendatud) oli vaja uurida võimalusi projekti ja tööde teostamise ja kontrollimise ühtlustatud meetodite rakendamiseks ning muldkeha tüüplahenduste kasutamise võimalikkuses .

Sõltuvalt regulatsiooni arengutest olid valikus kas Tee konstruktsiooni normdokumentatsiooni või juhiste muldkehaga seonduva ajakohastamine või teiste riikide vastavate juhendmaterjalide kasutamise soovitamine, et oleks tagatud võimalikult optimaalne tee terviklik konstruktsioon ja selle projekteerimise meetodid (tüüpkonstruktsioonide kasutamise võimalus) lähimate kümnendite piiratud ressursside (sh inimressursid) raames.

Käeolev aruanne on koostatud teostatud tööde loeteluna, millest olulisimad **on koondatud kolme juhise eelnõuna vormistatud dokumenti (LISAD 1, 2 ja 3) Arendusettepanekud on käesoleva aruande peatükis „Ettepanekud Transpordiameti juhiste muutmiseks viimaks tee projekteerimist Eurostandarditele vastavaks“.**

Eeltoodu Euroopa standarditega kooskõla tagamiseks on käeoleva uurimistöö vastutav täitja osalenud eksperdina Eesti standardite EVS-EN 16907 (Mullatööd) ja EVS-EN ISO 14689 ja 14688 (Pinnaste ja kalju liigitamine) tõlkimisel ning 2-e põlvkonna Eurokoodeks 7 (rakendamise töörühm) koostamise juures.

2 Tehtud tööde loetelu

Arendustöö algfaasis töötati läbi teede geotehnikaga seotud uuem teaduskirjandus, erinevad normid ja standardid, millest olulisematest koostati lühikokkuvõtted (lisad 4-6) mis kontsentreeruvad normides ja juhistes esitatud põhimõtteliste lahenduste toetamisele. Koostati ka erinevate riikide normide muldkeha osa ülevaade (lisa 7) Eeltoodu oli aluseks „Teede projekteerimise normide“ (TPN) uue redaktsiooni eelnõu koostamises Transpordiameti (TRAM) konsulteerimisele, eelnõu sõnastusettepanekute koostamisele jms. Kui TPN põhijooned olid selgumas, siis asuti eeltoodu baasil, arvestades võimaluse piires 2-e põlvkonna Eurokoodeksi uuendatud eelversiooni põhimõtteid, TRAM muldkeha geotehnilise projekteerimisega seotud juhiste ülevaatamist.

Uuema erialakirjanduse ja erinevate riikide juhiste läbitöötamise lõpptulemus arvestades lähteülesandes toodud suuniseid ja väljatöötamisel olevat TPNi ja 2-se põlvkonna Eurokoodeksit on koondatud järgmistesse 2021 aastal lõpliku versioonina valminud juhistesse (LISAD 1-3):

1. IPT Projektijuhtimine OÜ töö nr.19-05-1490/1. „Pinnasemudel teede projekteerimiseks. Geotehnilise uuringu ja pinnasemudelisse normsuuruste määramise juhend“, 2021.
2. IPT Projektijuhtimine OÜ töö nr.19-05-1490/2. „Tee muldkeha geotehniline projekteerimine. Tee projekteerimise normi ptk.5 kasutamise raamistik“, 2021.
3. IPT Projektijuhtimine OÜ töö nr.19-05-1490/3. „Geosünteetide kasutamine tee konstruktsioonis. Juhend“, 2021.
   Eeltoodute kohta eeltutvustusel esitatud arvamused on kommenteeritud Lisas 9.

Lisaks konsulteeriti tagamaks muldkeha geotehnilised lahenduste kooskõla Lepingus püstitatud ülesandega 2019-2021 aastal järgmiste ministri määrustena vormistatud või vormistamisel olevate dokumentide koostamist algfaasist kuni 2021 teise poolel ette valmistatud versioonideni:

1. Tee projekteerimise normid (MKM määrus nr 106 05.08.2015) muldkeha peatüki tehniline korrektuur 2021
2. Tee projekteerimise normid uusversiooni eelnõu 5 ptk väljatöötamine ning selle esimese versiooni 5 ptk kohta esitatud arvamuste läbitöötamisel ja arvamuste, ettepanekute koostamisel osalemine
3. Tee ehitamise kvaliteedi nõuded (MKM määrus nr 101 03.08.2015) kooskõla TPN muudatustega ja teiste riikide normide ning juhendmaterjalidega
4. Tee-ehitusmaterjalidele ja – toodetele esitatavad nõuded ja nende nõuetele vastavuse tõendamise kord (MKM määrus nr 74 27.09.2014)

Osaleti TallTech-i geosünteetide uuringu programmi koostamisel ja programmi läbi viimise tulemuste alusel koostatavate katsete protseduuride ette valmistamisel ning esialgsete rakendusettepanekute koostamisel:

1. Geosünteetide kiirendatud keemilise vanandamise protseduur
2. Geosünteetide roomekäitumise hindamise protseduur

3. Geosünteetide temperatuuri mõju hindamise protseduur
4. Protseduur geosünteetide roomekäitumise hindamiseks kombineerituna keemilise vanandamisega.

Koostatud on ettepanekud Transpordiameti juhiste muutmiseks viimaks tee projekteerimist Eurostandarditele sh Eurokoodeksile vastavaks (ptk 3) ja teostatud tööde kokkuvõte (ptk 4)

Koostatud on artikkel Teelehte Lisas 8.

3 Ettepanekud Transpordiameti juhiste muutmiseks viimaks tee projekteerimist Eurostandarditele sh Eurokoodeksile vastavaks

Kehtestada üldise raamistikuna järgmine dokument:

1. IPT Projektijuhtimine OÜ töö nr.19-05-1490/2. „Tee muldkeha geotehniline projekteerimine. Tee projekteerimise normi ptk.5 kasutamise raamistik“, 2021.

Vajalik seose loomiseks ja selgitamiseks Eurokoodeksi üldiste põhimõtetega.

Raamistik on koostatud selliselt, et on tagatud võimalus kasutada täies mahus (m106 muudatuse 2021 jõustumisel) seni kehtivat TPN muldkeha projekteerimise nõudeid, arvestades ka TRAM juhistes töö teostaja poolt ettenähtud muudatustega, samas võimaldab vajadusel täies mahus rakendada ka Soomes välja töötatud juhendeid tee projekteerimiseks, sh kataloogilahendid. Raamistiku muutmine ja täiendamine vajab tee geotehnikas pädeva isiku osalemist tagamaks geotehnilselt tervikliku ja Eurokoodeksi ja vastavate EVS-ISO-EN standarditega (kohustuslikku) kooskõla.

Kehtestada juhistena järgmised dokumendid:

1. IPT Projektijuhtimine OÜ töö nr.19-05-1490/1. „Pinnasemudel teede projekteerimiseks. Geotehnilise uuringu ja pinnasemudelisse normsuuruste määramise juhend“, 2021.
2. IPT Projektijuhtimine OÜ töö nr.19-05-1490/3. „Geosünteetide kasutamine tee konstruktsioonis. Juhend“, 2021.

Esimene neist on vajalik selleks, et geotehnilite uuringute tegemine muutuks eesmärgipäraseks, so. pinnasemudeli koostamiseks ja vajalike arvutuste tegemiseks normsuurustega iseloomustamiseks. Samuti sisaldab see juhised pinnaste ja materjalide kirjeldamiseks viisil ja moel, mis võimaldab nende klassifitseerimist mullatööde sh. tee muldkeha projekteerimiseks vajalikesse tüüpomadustega pinnaseklassidesse.

Teine keskendub eranditult geosünteetide kasutamisele tee konstruktsioonis jättes kõrvale sel eesmärgil tarbetud rakendused. Sisaldab endas projekteerimisjuhised ja näited ning materjalide pikaealisuse kontrolli meetodite loetelu.

Mõlema juhise muutmine ja täiendamine vajab tee geotehnikas pädeva isiku osalemist tagamaks geotehnilselt tervikliku ja Eurokoodeksi ja vastavate EVS-ISO-EN standarditega (kohustuslikku) kooskõla.

Erilahenduste projekteerimine

Täiendada loeteletud juhiste ja juhendite pealkirju alljärgnevalt:

1. Muldkeha ja dreenkihi projekteerimise, ehitamise ja remondi juhis. Erilahenduse projekteerimine (Maanteeameti peadirektori käskkiri nr.0001, 05.01.2016)
2. Elastsete teekatendite projekteerimise juhend. Erilahenduse projekteerimine (Maanteeameti peadirektori käskkiri nr.0088, 29.03.2017)
3. Katendiarvutus. Elastsete katendite arvutamise programmi kasutusjuhend. Erilahenduse projekteerimine. (MA 2017-002).

Muudatus on vajalik Eurokoodi üldiste põhimõtetega kooskõlla viimiseks ja selguse tagamiseks, et nimetatud dokumentide alusel toimub erilahenduse projekteerimine.

Täiendavalt tuleb TRAM juhendites üle vaadata kõik nõuded:

- mis on kehtestatud muldkehale, eriti töökihile ja filtratsioonimoodulile. TPN uues redaktsioonis ei nõuta kasutatud muldkehas pinnaste ja materjalide veejuhtivuse kontrollimist.

- Katendiarvutuse juures tuleb arvestada, et ainult eraldamise, filtreerimise ja dreenimise eesmärgil kasutatud geosünteedil puudub mõju tee kandevõimele. Geosünteetide kasutamisel saavutatav efekt konstruktsiooni kihtide paksuse vähendamisel ei ole lühiajaliste katsetamistega in-situ mõõdetav.

Kustutada juhisest „Elastsete teekatendite projekteerimise juhend“ (Maanteeameti peadirektori käskkiri nr.0088, 29.03.2017) peatükk 11.4 *Geosünteetide kasutamine teekatendi konstrueerimisel* täies mahus ja parandada vead.

Põhjendus:

IPT Projektijuhtimine OÜ töö nr.19-05-1490/3. „Geosünteetide kasutamine tee konstruktsioonis. Juhend“, 2021. juhendina kehtestamisel käsitletakse geosünteetide kasutamist oluliselt süsteemsemalt ja detailsemalt. Erineva käsitlemise detailsusega dubleerivate juhendite olemasolu on pigem eksitav. MA 2017-003 peatükk 11.4 on sisult informatiivne, mitte juhiseid sisaldav, samuti on vaja viia kooskõlla määrus 106 muudatusega, uuendada viited, freespuru kasutamine, parandada vead jne.

<u>Tüüplahenduste projekteerimine</u>

Täiendada loetletud juhiste ja juhendite pealkirju alljärgnevalt:

1. Muldkeha (remondi) projekteerimise juhis. Tüüplahenduste projekteerimine. (Maanteeameti peadirektori käskkiri nr.264, 29.12.2006)
2. Elastsete teekatendite projekteerimise juhend 2001-52. Tüüplahenduse projekteerimine.

Muudatus on vajalik Eurokoodeksi üldiste põhimõtetega kooskõlla viimiseks ja selguse tagamiseks, et nimetatud dokumentide alusel toimub tüüplahenduse projekteerimine.

„Elastsete teekatendite projekteerimise juhend 2001-52. kehtestada uuesti kui „Tüüplahenduse projekteerimine“

Muldkeha remondi projekteerimise juhis nimetada Muldkeha projekteerimise juhiseks.

Kataloogilahenduste projekteerimine

Enamus riike on teedeehituses enam kasutatavad lahendused arendanud välja kataloogilahendusteni. Kataloogilahendused on lihtsalt kohalikele pinnaseoludele kohaldatavad ja tee klassi arvestavad lahendused, mis on juhtivate inseneride poolt läbi arvutatud ja kontrollitud ning on külmakindlad ja piisava kandevõimega tüüppinnastest ja tüüpmaterjalidest koostatud lahendused.

Soovitame Eestis kasutusele võtta sobivad naaberriikides välja töötatud terviklikud katendi konstruktsiooni kataloogilahendused. Antud töö põhjal võib väita, et muldkeha projekteerimine ongi meie kehtiva normi järgi lahendatud tüüplahendusena ilma katalooglahendusi Vene normist üle võtmata. Senisele praktikale tuginedes võib Eesti väita, et Vene kataloogilahendusi (mis on olemas näiteks muldkeha soos vms) on Eesti mõistlik kasutada erijuhul.

Muud parandamist vajavad põhimõttelised vead

1. Muldkeha pinnaste tihendamise ja tiheduse kontrolli juhised (Maanteeameti peadirektori käskkiri nr.264, 29.12.2006)

    Kustutada punktid 2.10.2 kuni 2.10.4 kogu ulatuses. Ekspertiisiakt edastatud TRAMile ekspertarvamusena 16.09.2021.a.

Kokkuvõtvalt: Kõik eeltoodud juhised vajavad korrigeerimist ja uuendamist, teatud juhtudel nii kehtiva TPNi uuendamisega 2021 kui ka uue TPNiga kooskõlla viimist (tulenevalt erisustest mis on seotud TPN väljaspool 5 ptk nagu tee klassid kadumine jms), vigade parandamist, mille kohta on Tellijale teave edastatud ning tehnilist korrektuuri viimaks neid täismahus kooskõlla koostamisel olevate eelnõudega. Raamistikus on toodud alternatiivsed või Eestis puuduvad, kasutamiseks soovitatud Soome juhised, millest ei saa kasutada üksikuid osasid. Eesti juhistega Soome juhiste kooskasutamist, mille analoogid Eestis puuduvad, vajavad samuti olemasoleva Eesti juhiste (eeltoodud) korrektuuri.

4 Kokkuvõte

Lähteülesandes ettenähtud eesmärk on saavutatud. On leitud toimivad lahendused tee muldkeha ja veerežiimi projekteerimisel, ehitamisel ja järelevalvel, mis arvestavad võimaluse piires Eesti looduslikke ja Transpordiameti ning teostajate ressursse nii, et tee konstruktsiooni projekteerimine toimuks terviklikult, arvestades Euroopa ja maailma geotehnika arenguid (sh geosünteetika), samuti Eurokoodi 2-e põlvkonna hetkeseisu ja eriti selle 7 osaga seotud standardeid ning sh arvestades Eesti hetke nõudeid ja uut TPNi eelnõud, mis läbi antud arendustöö on kõige eeltooduga viidud hetkel kooskõlasse.

Eeltoodu rakendamiseks on töö tulemusel leitud, et Transpordiameti tee projekteerimist käsitlevad juhised ja juhendid ning vastavat tegevust reguleerivad ministri määrused on võimalik ja mõistlik ühtlustada Eurokoodeksi põhimõtetega läbi mullatööde standardi EVS-EN 16907 kirjeldatud pinnaseklasside ja neile omistatud tüüpomaduste.

Eestis aastakümneid katendiarvutuses kasutatud tüüppinnaste tüüpomadusi, kusjuures pinnaseklassidena võib EVS-EN 16907 mõistes lugeda GOST 25584 pinnaseid ja projekteerimise aluseks on neile „Elastsete teekatendite projekteerimise juhendis 2001-52“ omistatud tüüpomadused.

Transpordiamet on aastate vältel arendanud 2001-52. juhendit, täiendades juhendi 2017 aasta versioonis seda mitmete eripinnastega, mille omaduste määramine ei ole vastavuses tüüppinnastele esitatud nõuetega. Selliste pinnaste kasutamine on võimalik ka EVS EN 16907 raamistikus, kuid erilahendustena. Erilahenduste projekteerimine on reeglina keerukam, kuid tagab ökonoomsema lahenduse. Erilahenduse teostatavus tuleb enne ehituse algust katsetööga tõestada. Erilahenduse kasutamine ei ole mõistlik, kui sellega kaasnevad kulutused ei ole proportsionaalsed saadava majandusliku kasuga.

Tüüplahenduste ja erilahenduste kvaliteedi kontrollil ja töö vastu võtmisel on EVS-EN 16097 kohaselt teatud erisused, millega tuleb projektis ning tööde vastuvõtmisel arvestada.

Eestis puudub vajalik teadusbaas ja inimressurss oma kataloogilahenduste välja töötamiseks. Kataloogilahendused on kõiki teele esitavaid nõudeid täitvad läbi arvutatud lahendused tüüppinnastest ja tüüpmaterjalidest ning ei nõua keeruliste insenerarvutuste läbi viimist (arvutused on tehtud kataloogilahenduste välja töötamisel). Käesoleval hetkel puuduvad Eestis sellised kergesti rakendatavad ja kontrollitavad lahendused, ometi on enamus suuremaid riike valdavalt just kataloogilahenduste abil oma teid igapäevaselt projekteerimas.

2020 aasta lõpus pakkus Konsultant, et Lepingu teatud töödes oleks mõistlik Leping pikendada või teha paus, ootamaks ära uue teede projekteerimise normi ja Eurokood 7 sisuline valmimine, et sellega juhiste koostamisel arvestada. Kevadel 2021 selgus, et projekteerimisnorm jõustub kõige optimistlikuma stsenaariumi järgi 2022 teisel poolel, Eurokood 7 2023 alguseks. Tellijaga sai seda probleemi arutatud, vastav ettepanek tehtud ja alustati ka varakevadest konsultatsioone sel teemal, kuid tulenevalt positsioonist, millele Tellija juhtkond jõudis suve algul, ei peetud seda Tellija poolt võimalikuks. Eeltulenevalt on lepingujärgsed tööd viidud lõpule Lepingu kehtivuse perioodil arvestades teede projekteerimise normi eelnõu ja 2se põlvkonna Eurokoodeks 7 ettevalmistuse seisu 2021 esimesel poolel.

5 Ettepanekud edasiseks arendustööks

Uue tee projekteerimise normi TPN redaktsiooni jõustamisele eelnevalt tuleb üle vaadata senised TRAM juhendid, et tagada nende ühilduvus TPN-ga.

Ette valmistatavate Eurokoodeksi 2-e põlvkonna standardite vastu võtmisel tuleb üle vaadata senised TRAM juhendid, et tagada nende ühilduvus Eurokoodeksi uue redaktsiooniga.

Eurokoodeksi 2-e põlvkonna 7 seeria rahvuslik lisa vajab koostamist (senise lisa ülevaatmine ja vajadusel täiendamine ja muutmine). Samuti vajab Eurokoodeks 7 uusversioon korrektset tõlget ja tuleb hinnata Eesti oma rakendusjuhise loomise vajadust (analoog Soome NCCI 7-ga)

Käesoleva töö autorid on seisukohal, et hoolimata asjaolust, et koostatud raamistik võimaldab ühe võimalusena Vene projekteerimise süsteemi ja selle edasiarenduste baasil teede muldkeha (ja katendi) projekteerimise jätkamist, tuleks kaaluda sellest loobumist ja asendamist mõne teise, EL kuuluva naaberriigi projekteerimise nõuete ja lahendustega. Raamistik loob võimaluse Soome Väylävirasto juhendite ja juhiste koos InfraRYLiga terviklikuks kasutusele võtmiseks.

LISA 1

TEE MULDKEHA GEOTEHNILINE PROJEKTEERIMINE

TEE PROJEKTEERIMISE NORMI PTK 5 KASUTAMISE RAAMISTIK

Avaldamiseks kodulehel täismahus eraldi dokumendina.

Esitatud lõpparuande eraldi seisva osana. Kui antud töö alusel koostatakse ja avaldatakse Transpordiameti juhend, siis antud töö muutmine ja täiendamine saab toimuda pädeva isiku osalemisel tagamaks geotehnilist terviklikkust ja kohustuslikku kooskõla Eurokoodeksi ja vastavate EVS-ISO-EN standarditega. Muutmiste ja täiendamiste korral ilma IPT Projektijuhtimine OÜ kirjaliku kooskõlastuseta ei oma IPT Projektijuhtimine OÜ vastutust muudetud dokumendis toodud informatsiooni ja esitatud arvamuste eest, vaid see lasub täies mahus töö muutmise teostajal ja avaldajal.

Autor: Peeter Talviste

Sisu: Lepingu punkte 4.7-4.13 täitev ülevaatlik dokument, mis seob erinevad Eurostandardid ja Eestis ning Soomes kasutatava tee projekteerimise praktika. Dokument on koostatud selliselt, et on kasutusele võetav juhisena geotehniliste tööde projekteerimisel, vastu võtmisel ja kontrollil. 67 lk.

LISA 2

PINNASEMUDEL TEEDE PROJEKTEERIMISEKS

GEOTEHNILISE UURINGU JA PINNASEMUDELISSE NORMSUURUSTE MÄÄRAMISE JUHIS

Avaldamiseks kodulehel täismahus eraldi dokumendina.

Esitatud lõpparuande eraldi seisva osana. Kui antud töö alusel koostatakse ja avaldatakse Transpordiameti juhend, siis antud töö muutmine ja täiendamine saab toimuda pädeva isiku osalemisel tagamaks geotehnilist terviklikkust ja kohustuslikku kooskõla Eurokoodeksi ja vastavate EVS-ISO-EN standarditega. Muutmiste ja täiendamiste korral ilma IPT Projektijuhtimine OÜ kirjaliku kooskõlastuseta ei oma IPT Projektijuhtimine OÜ vastutust muudetud dokumendis toodud informatsiooni ja esitatud arvamuste eest, vaid see lasub täies mahus töö muutmise teostajal ja avaldajal.

Autorid: Pille Sedman ja Peeter Talviste

Sisu: Geotehniliste pinnaseuuringute läbi viimise ja pinnasemudeli koostamise põhimõtted tee muldkeha projekteerimiseks. Koosatud juhisena vajalike tööde tegemiseks. 38 lk.

LISA 3

GEOSÜNTEETIDE KASUTAMINE TEE KONSTRUKTSIOONIS

Avaldamiseks kodulehel täismahus eraldi dokumendina. Esitatud lõpparuande eraldi seisva osana. Kui antud töö alusel koostatakse ja avaldatakse Transpordiameti juhend, siis antud töö muutmine ja täiendamine saab toimuda pädeva isiku osalemisel tagamaks geotehnilist terviklikkust ja kohustuslikku kooskõla Eurokoodeksi ja vastavate EVS-ISO-EN standarditega. Muutmiste ja täiendamiste korral ilma IPT Projektijuhtimine OÜ kirjaliku kooskõlastuseta ei oma IPT Projektijuhtimine OÜ vastutust muudetud dokumendis toodud informatsiooni ja esitatud arvamuste eest, vaid see lasub täies mahus töö muutmise teostajal ja avaldajal.

Autorid: Pille Sedman ja Peeter Talviste

Sisu: Geosünteetide kasutamise põhimõtted tee projekteerimiseks. Koosatud juhisena projekteerimiseks koos arvutusnäidetega. 48 lk koos lisadega.

LISA 4

LÜHIKOKKUVÕTE

FHWA NHI-05-037 Geotechnical Aspects of Pavements (2006)

Chapter 1 Introduction

Pavement system

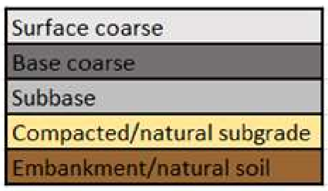

Figure 1. Pavement system

Pavements system provides a smooth surface for safe passage of vehicles under all weather conditions and for a predetermined period.

Subgrade- the top layer of natural soil which the pavement system is constructed on and it may be compacted or stabilized.

Subbase- A layer (or layers) of specified materials to support the base course. Lower quality (and less expensive) than base and is included typically when subgrade soils are of poor quality. Secondary functions: prevent gradation, minimize frost action, provide drainage and provide working platform for base construction in case of poor soils.

Base- A layer (or layers) of specified material placed on subbase or subgrade to provide stable and uniform support for surface course. Typically of higher quality materials than subbase. Same secondary functions as subbase.

Surface course- A layer (or layers) designed to accommodate traffic load. Must resist traffic abrasion (hõõrdumine) and skidding (libisemine), climate and minimize water infiltration. Typically consisting of asphalt ("flexible" pavement) or Portland cement concrete ("rigid" pavenement).

Pavements system could additionally contain drainage systems and/or filter materials (e.g. geotextiles).

Typical pavement types

Rigid pavement- surface course is Portland cement concrete (PCC) constructed on granular base layer on subgrade soil. PCC is the primary load carrying component. Gradation characteristics of base and subbase are critical. PCC slabs can be reinforced with steel strands for better stress resistance. Air-entrainment increases thaw and freeze resistance.

Flexible pavement- has an asphaltic surface layer. Relies on strength and stiffness of underlying layers to supplement the load carrying capacity of asphalt. Hot mix asphalt concrete (AC) is most commonly used and must be compacted (up to compaction levels of 92%) during construction.

Composite pavement- asphaltic surface placed on PCC or bound base.

Unpaved or naturally surfaced roads- Not paved roads which rely on granular layers and subgrade for load carrying capacity. Constructed in sparse population areas with low volumes of traffic.

According to construction type, pavements can be a new construction, rehabilitation or reconstruction. New construction is pavement construction on a non-paved alignment, rehabilitation is the repair/upgrading of some of the pavement layers and reconstruction is the complete removal of existing pavement and replacing it with a new one.

Pavement performance and geotechnical issues

Failure is a pavement section that the is extensively cracked or rutted and more than was anticipated during pavement design. Collapse or breakdown of any of the pavements components is termed structural failure which renders the pavement unable to sustaining loads. Functional failure occurs due to increased roughness and causes discomfort to passengers and high stresses on vehicles. Typically, the causes for failure are inadequate maintenance, excessive loads, climatic (freezing, drying), poor drainage causing poor subgrade conditions, intrusion of subgrade materials to granular base, improper construction practices. Visible pavement damage (e.g. cracks, rutting) and ride quality can be caused by deficiencies in the surface layers of AC or PCC but could also be in part due to failures in the underlying unbound layers.

The most important geotechnical variables causing distresses and failure are stiffness and strength of the unbound pavement layers. The strength indices, the California bearing ratio (CBR) or the resilient modulus (M_R) can be used to characterize the material quality of these layers. However, the typical stresses that develop in pavements are well below the failure threshold of the unbound materials. In reality the cause of failure and distresses are degradation of the strength/stiffness of the unbounds layers due to moisture/drainage, freeze/thaw, intermixing of layers and erosion.

Permanent deformations are most typically *ruts* on the surface of AC in flexible pavements. In case of a well-designed pavement about 1/3 of the rutting observed on the surface occurs in the AC layer while the other 2/3 is due to accumulated deformation of geomaterials in the base, subbase and subgrade layers (caused by factors given above e.g. freeze/thaw etc). *Fatigue cracking* occurs due to repetitive loading and is governed in both rigid and flexible pavements by the fatigue resistance of the surface layer and cyclic tensile strain at the bottom of the layer. The latter is strongly affected by the subbase and subgrade prone to above listed causes for failure. *Reflective cracking* occurs in AC or PCC due to cracks in sublayers with the same causes for failure as counted above. *Potholes* are caused by excessive stresses in the surface course layer over voids in the base and subbase caused primarily by freezing. Pavement surface *roughness* is impacted by same geotechnical factors as ruts.

During the design of any pavement an anticipated service lifetime should be anticipates after which the pavement would require repairs or replacement and normal rutting during that period should be anticipated to be below 20- 25 mm. However, premature pavement failures are primarily by caused by geotechnical issues as listed above and not taking them into enough consideration during pavement design.

Chapter 3 Geotechnical issues in pavement design and performance

The satisfactory performance of the pavement during its lifetime is achieved with an integral view of the pavement system with all of its key components. The key components are the wearing surface, the bound structural layers, unbound base and subbase layers, the subgrade, additional drainage and remedial measures (e.g. the use of geosynthetics).

The unbound layers of the pavement system can provide over 50% of the total structural capacity of pavement systems. The stresses induced by traffic loads on pavement are highest in upper layers and decrease with increasing depth therefore higher quality (more expensive) material are used in the upper layers. Material usage optimization could therefore offer great advantage for construction cost. This also requires greater attention to lower layers since the upper layers could be rehabilitated (and avoid large expenses of total reconstruction) if the lower layers perform well over long-term.

An important parameter affecting pavement systems is moisture (and other environmental factors) which has detrimental effects on the pavement system reducing stiffness and strength. Additionally, in cold environments freeze/thaw cause swelling and mixing of granular sizes in unbound layers.

General issues

The pavement design whether it is new construction, rehabilitation or reconstruction impacts key geotechnical aspects. New constructions require comprehensive on-site exploration and material characterization of the surface materials. For rehabilitation designs the background information is available from original records and only minor characterization of sub-layers which remain part of the new pavement system is needed. Reconstruction designs also typically use original geotechnical data since new tests on soils can only be performed once the old pavement system is removed. Pavements constructed on natural subgrade require extensive exploration and material characterization. Additionally, when cuts and fills are used attention must be paid to transition zones (nonuniform support and drainage). Ground-water monitoring is advised for cuts.

It is well known that high moisture content decreases pavement service time even more so with cycles of freezing/thawing. Water can enter the pavement system through cracks on the surface, by capillary pressure from the bottom or from the pavement sides. Moisture and temperature greatly influence the properties of various pavement components such as changes in the modulus of asphalt bound materials with temperature; stresses in cementitious materials; changes of resilient modulus of unbound materials due to freezing and reduced strength after thawing; moisture decreased modulus of coarse grained layers and affect the structure of the soil. Three approaches are to prevent moisture from entering the pavement system, use designs and materials which are insensitive to moisture; quickly remove the water from the pavement system.

In cases of unsuitable natural soils at the construction site methods such as blending or mechanical stabilization (compaction) or mixing of stabilizing admixtures or geosynthetics can be applied to enhance the properties and potentially cut expenses.

Geotechnical factors

Geotechnical materials which are of interest for pavement design include non-stabilized granular base and subbase materials, non-stabilized subgrade soils and mechanically/chemically stabilized subgrade soils and bedrock groups. The properties of these materials include physical properties (density, moisture content), stiffness and strength, thermo-hydraulic properties (drainage) and performance related properties. Stiffness as characterized by the resilient modulus is the most important geotechnical property.

Pavement design is sensitivity to the geotechnical factors in terms of the overall cost of the construction. The strength and stiffness of the subgrade soil (inputs into structural design) determine the thickness and quality of the base/subbase layers to achieve a desired load carrying capacity. The base/subbase effectively determine the cost of the project e.g. a reduction of California bearing ratio (CBR) from 8 to 4 results in about 20% increase of the project costs. This also suggests that in some cases stabilization/compacting of the subgrade may be more cost efficient than increasing base/subbase layer thickness or quality.

Pavement design utilizes 3 approaches: empirical, mechanistic and empirical-mechanistic. Empirical design is based on experimental results and experience and are not backed by rigorous scientific models. They are simple to apply with "real-world" data but are often limited to specific conditions e.g. in one geographical region. The mechanistic approach utilizes theories of mechanics to relate pavement structural behavior to loads based mainly on elasticity theory. To date a fully mechanistic theory does not exists and some empirical inputs are required which leads to the empirical-mechanistic approach which combines e.g. calculation of pavement stresses and relates them with empirical distress models.

AASHTO pavement design guide

AASHTO pavement design guide is based on empirical methods based on field test. The Pavement Serviceability Index (PSI) characterizes the overall qualitative condition of the pavement with the primary determining factor being roughness. The design relates traffic with pavement structure and performance through empirical equations (3.1- 3.9) for both rigid and flexible pavement systems.

The structural number (SN) of flexible pavements defines the main geotechnical inputs of the AASHTO guide through the equation $SN=a_1D_1+a_2D_2m_2+a_3D_3m_3$ (a- layer coefficient; D- layer thickness; m- drainage coefficient and 1, 2, 3 refer to the surface, base and subbase layers). The coefficients a_2 and a_3 are calculated from the base and subbase resilient moduli E_{BS} amd E_{SB}. The final geotechnical input is the resilient modulus M_R of the subgrade which characterizes the soil support and is a measure of the elastic stiffness.

In case of rigid pavements, the primary input parameter is the thickness of the concrete slab D. Other required inputs are subgrade reaction $k=f(_{MR}, E_{SB})$, subbase thickness D_{SB} and depth to rigid foundation D_{SG} and loss of support factor LS. In the final version of AASHTO guide for both pavements types seasonal variation of input parameters can be taken into account.

The sensitivity of SN and cost for flexible pavements and slab thickness for rigid pavements varies differently for various soil parameters. For flexible pavements structural number and cost are sensitive to soil support (and M_R) but not so much to environmental conditions. For rigid pavements on the other hand the slab thickness D is insensitive to foundation stiffness (subgrade reaction) and moderately sensitive to M_R and LS. The drainage coefficient strongly affects the required base thickness and cost index in case of flexible pavements and required slab thickness in case of rigid pavements.

The AASHTO guide does not take into account climatic variations, heavy traffic loading and rehabilitation limitations. These limitations are addressed with NCHRP 1-37A mechanistic-empirical approach, but its implementation requires more input data, higher computational power and software and expertise.

Low-volume roads are designed primarily as aggregate of natural soil roads. The main parameter to consider is the rutting depth which is caused by compressive and shear stresses from vehicle loads. Sever rutting reduces the road serviceability and vehicle speeds. The extent of rutting depends on the soil support characteristics and is most commonly characterized with the CBR.

Chapter 4 Geotechnical exploration and testing

Geotechnical exploration aims at giving an understanding of the subgrade as an input for pavement design and its influence on pavement construction and performance. The investigation may be conducted by a variety of complementary methods. In many cases simplistic approaches such as SPT testing do not give enough data required for adequate design thus resulting in over- or under conservative designs. This can either result in excessive costs or premature failure. Although, the cost of extensive geotechnical exploration is debated it is several orders of magnitude less than are materials used in excess or reconstruction. The sampling is in most cases one in millionth or more while for concrete/asphalt it Is 1-100000 although the material properties of the subgrade vary considerably more. Attention should also be paid to data from alternate available sources e.g. initial records in case of rehabilitation/reconstruction. Additionally, alternate methods prior to drilling can give information to optimize boring and sampling locations. The design steps should be taken as follows: establish pavement type, search available data, site pre-exploration, exploration plan with adequate methods, conceptual design and finally lab testing of representative soils.

Levels of geotechnical exploration for different construction types

For **new constructions** a complete exploration of subgrade is necessary together with evaluation of base/subbase layers with materials available near construction site. Prior to subgrade exploration program, identification of geological features and available site information (site reconnaissance, air photos, geological maps) together with the initial design criteria for the pavement should be analyzed. The input geotechnical parameters (e.g. M_R) for design need to be determined (*in-situ* or lab test) together with identification and classification of subgrade soils to identify vertical and horizontal variability of subgrade properties. Lastly, availability of materials at the construction site and the groundwater table need to be determined.

For pavement **reconstruction** already existing information (e.g. borings data) should be carefully analyzed regarding the design requirements of the new pavement. Distresses in the existing pavement should be evaluated and limited additional subgrade testing is advised. Acquiring M_R and CBR values for existing base/subbase and subgrade layers could be needed due to deteriorated properties (strength/stiffness) of these layers e.g. due to excessive moisture content. The value of reworking the subgrade can be determined from comparing M_R values of compacted and natural samples of the subgrade material at specific moisture content. Subsurface investigation can be typically performed with non-destructive methods such as the FWD which yields M_R for back-calculation of the elastic moduli of the *in-situ* soils. Additional in-situ test can be performed (e.g. dynamic cone penetrometer (DCP), field CBR) to asses subgrade strength.

Pavement **rehabilitation** requires different geotechnical exploration based on different factors: pavement condition, distress type and severity, rehabilitation techniques, possible pavement upgrades and performance. Similarly to reconstruction available information should be assessed with additional borings and corings. Geophysical tests (FWD, GPR) can help locate problematic areas in the subgrade which should then be more thoroughly investigated with borings. M_R values from e.g. FWD can be used to back-calculate the elastic modulus and strength properties of the subgrade and base/subbase layers. Sub-surface sampling can validate these properties.

Identification of available information

A thorough investigation of the available information to identify where the subgrade starts and what soils to anticipate will help to identify the extent and type of necessary exploration program. Furthermore, prior research can help identify key locations for borings and greatly enhance the quality of the subsurface exploration (e.g. different frequency of borings in ancient lakebed vs highly variable geology). Information about soil deposition process can be gained from characteristic topographic features called landforms. Soils can further be classified into residual or transported with typical properties (table 4-1). Consulting national soil maps (if available) can yield useful data.

Site reconnaissance

A visit to the construction area together with the project plans is the next step to further increase the knowledge before planning the subsurface exploration program. Potential information includes: general site conditions, surface geological features, type of existing pavement (if any), possible traffic control requirements, adjacent land use, environmental issues, working hours, boring limitations, flod levels, subsurface conditions from exposed cuts, surface settlement.

Subsurface exploration program

The field exploration methods, sampling requirements and frequency of test should be determined from the gathered information. The aim of the subsurface exploration is to obtain sufficient data for selection of principal types, locations and dimensions of pavement foundations, provide adequate cost estimation and identify the site in sufficient detail. Soil and rock subsurface conditions need to be defined:

- Areal extent of subgrade features: physical description, extent, thickness and top and bottom of each stratum.
- For cohesive and granular soils, the identification of soils in each stratum and assessment of pavement support and possible construction issues (e.g. stabilization requirements).
- Groundwater for each aquifer within zone of influence of pavement
- Bedrock and presence of boulders within zone of influence of pavement

The depth to which the subsurface conditions influences the pavement design, construction and performance should be evaluated for correct subsurface exploration. The AASHTO guide suggests 1.5 m depth for typical condition and a rule of thumb suggests two times the width of load i.e. typically 1.5- 2m. From the perspective of pavement design the critical layers are in the upper 1m. However, high groundwater or bedrock (within 3 m of the surface) can critically influence pavement deflection and frost susceptibility. Therefore, a limited number of borings should always be performed to depths of 6 m.

The techniques used for subsurface exploration includes remote sensing, geophysical and *in-situ* investigations and borings and sampling. **Remote sensing** data from satellites and air photo can be used to identify terrain conditions, geological formations, escarpments, buried stream beds and general soil/rock conditions. **Geophysical investigations** can help selectively identify borings, supplement borehole data and extrapolate between borings. Measurements include both mechanical waves (deflection response, seismic refraction, crosshole, downhole and spectral analysis of surface wave tests) and electromagnetic EM techniques (resistivity, EM magnetometer and radar). Elastic propertied of the subsurface can be determined from mechanical waves. The advantage of geophysical tests are: non-destructive and non-invasive, fast and economical, provide theoretical background for interpretation and can be applied to soils and rocks. They however do not provide any soil samples and interpretation can be complicated without supplementing tests.

The most often method is falling weight deflectometer (FWD) and lightweight deflectometers (LWD) which can help determine subgrade properties especially well for reconstruction and rehabilitation processes. High-speed deflectometry enables tests without traffic control at a speed of ca 3-20 km/h and substantially increases the amount of information gained.

EM methods such as surface resistivity (SR), ground penetrating radar (GPR), electromagnetic conductivity (EM) and magnetic survey (MS) are based on measuring the resistivity of the surface (or conductivity of pore water in soil and rock). Mineral grains of soil and rock are non-conductive. Data from EM methods such as the EM can be correlated with the soil type and clay content. EM methods allow for mapping of the entire construction site and offer possibility to discover subsurface anomalies, boulders etc.

***In-situ* testing** (penetration type and probing type) can supplement soil borings and compliment geophysical results. Effective and cost-effective when made with conventional sampling. *In-situ* tests used to rapidly evaluate the variability of subgrade support conditions, locate regions that require more sampling and identify rock and groundwater. While SPT is the most commonly used method, DCP and CPT offer more reliable results and more efficient and rapid sampling. The DCP and CPT provide data on subsurface soils without sampling disturbance in real time and provide strength characteristics of the subgrade. DCP is more qualitative but also more rapid and is excellent for rehabilitation and reconstruction projects. The CPT provides quantitative results directly correlated with soil properties and inputs for pavement design.

Boring and sampling is the most complex and expensive part of the exploration program therefore careful planning of the program is advised. It is used to collect disturbed and undisturbed samples for determination of engineering properties of soils in the lab. Care must be taken to avoid contamination of layers in undisturbed soils. In typical pavement design a subgrade support value is assigned to a lengths of 1- 16 km but this requires verification of uniform subgrade conditions both vertically and horizontally. Local variations in support condition can be included with additional bore holes.

The to approaches are systematic and representative sampling. Systematic sampling is done at uniform horizontal and/or vertical intervals. In case of varying conditions between two bore holes an additional one is made in the middle. A large sample size is obtained. Representative approach is based on the engineering judgement based available info, remote sensing and geophysical tests and on what are the representative soils and subgrade support values. The frequency and depth of boring is again dependent on the available info, remote sensing and geophysical tests and local soil conditions. Lack of such test requires more extensive boring program. Interval can vary from 2 to 12 borings per kilometer. In general, the boring frequency should be higher for new constructions that rehabilitation/reconstruction. It is to be noted that the cost of a few extra boring in negligible with premature failure of the pavement system. In case of rehabilitation boring should be placed in the wheel path. Typical depth of borings should be 1.5- 2m below the proposed pavement subgrade elevation (excluding to be removed soft soil) but several borings should be down to 6 m to determine water level and bedrock.

Sampling of soil will vary with type of pavement project. New constructions require more sampling and thus the samples are mostly be disturbed. Rehabilitation/reconstruction typically require detailed sampling of the base/subbase and subgrade layers but overall much less samples are needed. Sampling can be continuous or intermittent with the latter requiring probes at least every 1.5 m and at every change of stratum. The knowledge of engineering characteristics (shear strength, consolidation characteristics, M_R) requires undisturbed sampling. The storage of samples should not take place in extreme condition (e.g. freezing temperature) and should be in moisture tight sampling containers/bags.

The need for rock sampling depends on location of the bedrock, geology of the region and availability of local data. The main consideration is that rock above subgrade must be removed by ripping or blasting. Latter is 4 to 20 times more expensive and can cause other issues (e.g. noise). Refraction survey methods can detect ripability and SPT values of 80 to 100 is the upper limit for ripping. The proximity of the bedrock to the subgrade dictates additional samples. The rock surface must be level and prevent water trapping in local depressions. Rock cores should be obtained to a depth of 0.6-1 m to identify, determine the rock quality and durability.

Observations of groundwater are an important part of geotechnical exploration. Groundwater level is an input in the mechanistic-empirical design process and will also influence the interpretation of FWD results. Measuring water entry at drilling measuring groundwater level after drilling is the minimum to determine groundwater level but can be complemented with additional test (e.g. test wells).

Test pits allow for detailed examination of the subsurface strata and are relatively inexpensive at shallow depth which should generally not exceed 2- 3 m.

Samples are also required from materials (preferably granular) used as fill and borrow sources. Moisture-density relationship and required compaction should be determined.

Geophysical exploration methods

Falling weight deflectometer (FWD) is used to determine variations in pavement layer, subgrade stiffness, back-calculate M_R and to identify area which require more sampling along the length of the pavement. FWD is based on dropping a falling weight (typically 4- 107 kN) onto the pavement and measuring the vertical deflection of the pavement at various distances from the center of the load. The apparatus is typically mounted on a trailer and measurements are taken approximately every 20-50 m to yield deflection profiles along the length of the pavement. The profiles are used to back-calculate the effective M_R of various pavement layers and subgrade using a multi-layer linear elastic model. The FWD simulates a moving wheel and can be used for all construction types. The influence depth may extend to 9 m and care is needed for interpretation of results. Its advantages are speed, repeatability, non-destructive, direct evaluation of M_R, equipment robustness, ease of transport and simulating of moving wheel. Its disadvantages are traffic control requirement, static tests require stopping for measurements, requirement for well defined layer thickness and that deep features can affect results and M_R values are often over-predicted.

Surface resistivity (SR) is used to locate bedrock, stratigraphy, wet regions, compressible soils, karstic features, contamination plumes, buried objects and map faults. The electrical resistivity is the reciprocal of electrical conductivity and is measured in ohm-meters. In general, resistivity increases with soil grain size. SR is sensitive to moisture content and can be used to map variations in moisture. Its advantages are moderately fast (150m/h), simple, can evaluate significant depths, works for sublayers with lower/higher resistivity and automation. Its disadvantages are traffic control requirement, testing of asphalt/concrete requires coring, lateral resistivity variations affect results and metal objects in ground affect results.

Ground penetrating radar (GPR) can be used to define subsoil strata, moisture variation, depth to rock, voids, buried pipes and cables, determine thickness of various layers and characterize archeological sites. GPR measurements are performed with air or ground (for more depth) transmitter-receiver antennas and measuring the change in the short impulses of high-frequency electromagnetic waves. Changes in the soil permittivity reflect changes in the subsurface environment. Its advantages are fast (2- 80 km/h) and easy to use, varying antennas varies measurement depth, non-destructive and real-time continuous subsurface data. Disadvantages are traffic control requirement, restricted depth in saturated soil clays and difficult interpretation.

Electromagnetic conductivity (EM) is excellent in identifying clay, locating metal objects, map geological units and groundwater contaminants typically within 1-2 m (can be up to 5m). The method is based on electromagnetic induction by using two coplanar coils. First coil that produce a secondary time-varying (typically kHz) current in the soil which is then detected with the second coil. Measurements are affected by saturation, porosity and salinity of the pore fluids. Its advantages are relative fast (2-8 km/h), data recording is easy and efficient, instruments can be tailored to use at different depths. Disadvantages are traffic control requirements, may require several passes and it is influenced by metal objects.

Mechanical wave using seismic refraction are used locate depth and characteristics of bedrock and to evaluate dynamic elastic properties of soil and rock. The method involves generating seismic waves that refract from different subsurface layers and are detected with linear sensors. Seismic energy travels with velocity which is dependent on layer properties such as density, porosity, structure and moisture content. The method is aimed at quick and economical extending of subsurface exploration of large areas. Also used to determine rock rippability. It advantages are lightweight equipment (2 man crew), very effective at locating bedrock, well-established correlation with rippability and can be used where drilling is not possible. Its disadvantages are slow, traffic control requirement, lateral deposition can influence results and interference from seismic noise.

In-situ **methods**

Standard penetration test (SPT) is a quick means to evaluate variability of subgrade with correlation to density of granular soils and obtain disturbed samples. The number of blows (from a 63.5 kg hammer falling from 0.76 m) to advance a thick wall tube into the ground by 300 mm is measured. Most important factor is the energy efficiency of the system which varies with equipment and operator and therefor calibration is advised. SPT can be performed with a variety of soil types and weak rocks but not particularly useful for gravel deposits and clays. SPT correlates with angle of internal reflection, undrained shear strength and modulus but due to large scatter should not be used solely for design inputs. It provides subsurface variability and can help identify locations of extra borings and data from SPT can be reasonably compared with FWD data. Its advantages are to obtain both sample and number, simple, suitable in many soil types and weak rocks. Its disadvantages are collecting simultaneously of sample and number is poor accuracy, disturbed samples, a crude number for analysis, not applicable in soft clays and soils and high variation and uncertainty.

Dynamic cone penetrometer (DCP) is used to determine in place strength of fine- and coarse grained soils. A direct correlation exists between soil strength and penetration with a solid object such as a cone. A cone is driven into the ground with a hammer (4.6- 8 kg). Its advantages are easy operation, easy site access, simple equipment, immediate results, usable in pavement core holes, suitable in many soil types (including weak soils) and fairly reliable. Its disadvantages are high uncertainty in gravel soils, does not obtain a sample, index tests only, limited depth to 1 m and difficulties in cone extraction.

Cone penetrometer test (CPT) provides a fast and economical method for continuous profiling of soil properties and geo-stratigraphy. The test consists of measuring the resistance of the soil to penetration from a steel rod driven into the ground at constant velocity. Requires depth measurement with e.g. a potentiometer. It is suitable from soft soils and clays to dense sands but is not appropriate in gravel and rocky terrain. Complements SPT well. Its advantages are fast and continuous profiling, economical and productive, strong theoretical basis, not operator dependent, very suitable for soft soils and good reliability. Disadvantages are high investment, skilled operator required, no soil samples, not suitable for gravel and electrical drift, noise and calibration.

The subsurface exploration program should start with remote survey, followed by FWD that would map the area for geotechnical features and determine M_R so as to reduce the number of borings. CPT and DCP would then be used to classify soil strata and strength values and thickness profiles. Finally, boring in predetermined locations. Such a program is aimed at developing a detailed understanding of the subgrade and to be cost effective. Such an extensive program has been developed in Finland where philosophy is that subgrade should last for 60 – 100 years, base/subbase for 30- 50 years and surface layers for 15- 20 years.

Subgrade characterization

After evaluating conceptual design, determining other geotechnical features, subsurface drainage and subgrade stabilization and identifying construction materials the final step is the subgrade characterization through evaluation of field data and classification test, stratigraphic profiles and selecting representative soil layers for testing.

Boring logs is the basic record detailing every geotechnical exploration. It provides both detailed information about the tests, groundwater level, boring locations, soil types and properties but also background data on the project. A boring log should include: topographic survey data with boring locations and elevation data, identification of subsoils and bedrock, thickness of each layer, sampling and drilling data, water level observations and closure of borings. Note should be taken whether soil classification was done visually on the field or in the lab.

Subsurface profiles should be constructed in the pavement longitudinal alignment with limited number of transverse profiles. Subsurface data should additionally be presented as a map with general trends in the area of the project. However, due to the requirement for interpretation neither of the documents should not be included in the investigation report nor the bid documents.

A program of lab test will be required on representative samples from the construction site. The extent of the program depends on the criticality of the design and on complexity of soils. Primary test should be M_R and CBR tests and if possible should be performed on undisturbed specimen. The number of test specimens depends on the number of identified soil types. Most of the tests should focus on samples from the 0.6 m below the planned subgrade with some tests involving greater depths. Variability tests should be included.

Chapter 5 Geotechnical inputs in pavement design

The most important geotechnical inputs for pavement design are those related with material properties of unbound layers and subgrade soils. Additionally, the influence of climate e.g. freeze/thaw cycles on the behavior of unbound cycles in critical. Design inputs vary between the empirical AASHTO design and the mechanistic-empirical design (NCHRP 1-37A).

1. Required geotechnical inputs

The AASHTO design guide involves geotechnical inputs separately for flexible (5-1) and rigid (5-2) pavements and additionally includes environmental parameters (e.g. swelling and frost heave).

Table 5-1. Required geotechnical inputs for flexible pavement design using the 1993 AASHTO Guide.

Property	Description	Section
M_R	Resilient modulus of subgrade	5.4.3
E_{BS}	Resilient modulus of base (used to determine structural layer coefficient)	5.4.3
m_2	Moisture coefficient for base layer	5.5.1
D_2	Thickness of base layer	
E_{SB}	Resilient modulus of subbase (used to determine structural layer coefficient)	5.4.3
m_3	Moisture coefficient for subbase layer	5.5.1
D_3	Thickness of subbase layer	
θ	Swell rate	5.6.1
V_R	Maximum potential swell	5.6.1
P_S	Probability of swelling	5.6.1
ϕ	Frost heave rate	5.6.1
ΔPSI_{MAX}	Maximum potential serviceability loss from frost heave	5.6.1
P_F	Probability of frost heave	5.6.1

Note: Additional sets of layer properties (E_i, m_i, D_i) are required if there are more than two unbound layers in the pavement structure (exclusive of the natural subgrade).

Table 5-2. Required geotechnical inputs for rigid pavement design using the 1993 AASHTO Guide.

Property	Description	Section
M_R	Resilient modulus of subgrade	5.4.3
E_{SB}	Resilient modulus of subbase	5.4.3
D_{SB}	Thickness of subbase	
D_{SG}	Depth from top of subgrade to rigid foundation	
LS	Loss of Support factor	5.4.6
C_d	Drainage factor	5.5.1
F	Friction factor (for reinforcement design in JRCP)	5.4.7
θ	Swell rate	5.6.1
V_R	Maximum potential swell	5.6.1
P_S	Probability of swelling	5.6.1
ϕ	Frost heave rate	5.6.1
ΔPSI_{MAX}	Maximum potential serviceability loss from frost heave	5.6.1
P_F	Probability of frost heave	5.6.1

The NCHRP 1-37A design guide requires substantially more input information characteristic of mechanistic-empirical design. It utilizes a hierarchical approach based on the importance of the project which determines the required design inputs. *Level 1* inputs provide highest accuracy (lowest uncertainty) and is implemented for pavements with high volumes of traffic and/or safety or economic concerns in case of early failure. *Level 2* inputs provide intermediate level of accuracy (comparable to AASHTO guide) and is utilized when the project is not so critical or in case of limited test equipment and subsurface exploration program. *Level 3* inputs lowest level of accuracy utilized in projects with minimal consequence of early failure (e.g. low traffic). Inputs are typical regional values. The benefits of level 1 design (compared to cost) are however hard to quantify. The input level may also vary for the same project for different parameters. The hierarchical approach provides great flexibility, allowing for initial test programs to be upgraded with time and its concept allows to find the best accuracy vs cost design. Geotechnical inputs are classified as mechanical, thermo-hydraulic and distress model.

Table 5-3. Geotechnical mechanical property inputs required for the flexible pavement design procedure in the NCHRP 1-37A Design Guide.

Property	Description	Level 1	Level 2	Level 3	Section
General					
	Material type	✓	✓	✓	3.3.2
γ_t	In-situ total unit weight	✓	✓	✓	
K_0	Coefficient of lateral earth pressure	✓	✓	✓	5.4.9
Stiffness/Strength of Subgrade and Unbound Layers[a]					
k_1, k_2, k_3	Nonlinear resilient modulus parameters	✓[b]			5.4.3
M_R	Backcalculated resilient modulus	✓[c]			5.4.3
M_R	Estimated resilient modulus		✓[d]	✓	5.4.3
CBR	California Bearing Ratio		✓[d]		5.4.1
R	R-Value		✓[d]		5.4.2
a_i	Layer coefficient		✓[d,e]		5.4.5
DCP	Dynamic Cone Penetration index		✓[d]		4.5.5
PI	Plasticity Index		✓[d]		5.3.2
P200	Percent passing 0.075 mm (No. 200 sieve)		✓[d]		5.3.2
	AASHTO soil class			✓	4.7.2
	USCS soil class			✓	4.7.2
ν	Poisson's ratio	✓	✓	✓	5.4.4
	Interface friction	✓	✓	✓	5.4.7

[a]Estimates of M_R and ν are also required for shallow bedrock.
[b]For new construction/reconstruction designs only.
[c]Primarily for rehabilitation designs.
[d]For level 2, M_R may be estimated directly or determined from correlations with one of the following: *CBR*; *R*; a_i; *DCP*; or *PI* and *P200*.
[e]For unbound base and subbase layers only.

Table 5-6. Distress model material properties required for the NCHRP 1-37A Design Guide.

Property	Description	Level 1	Level 2	Level 3	Section
k_1	Rutting parameter (Tseng and Lytton model)	✓	✓	✓	5.4.8

Table 5-4. Geotechnical mechanical property inputs required for the rigid pavement design procedure in the NCHRP 1-37A Design Guide.

Property	Description	Level 1	Level 2	Level 3	Section
General					
	Material type	✓	✓	✓	3.3.2
γ_t	In-situ total unit weight	✓	✓	✓	5.3.2
K_0	Coefficient of lateral earth pressure	✓	✓	✓	5.4.9
Stiffness/Strength of Subgrade and Unbound Layers[a]					
$k_{dynamic}$	Backcalculated modulus of subgrade reaction	✓[b]			5.4.3
M_R	Estimated resilient modulus		✓[c]	✓	5.4.3
CBR	California Bearing Ratio		✓[c]		5.4.1
R	R-Value		✓[c]		5.4.2
a_i	Layer coefficient		✓[c]		5.4.5
DCP	Dynamic Cone Penetration index		✓[c]		4.5.5
PI	Plasticity Index		✓[c]		5.3.2
P200	Percent passing 0.075 mm (No. 200 sieve)		✓[c]		5.3.2
	AASHTO soil class			✓	4.7.2
	USCS soil class			✓	4.7.2
ν	Poisson's ratio	✓	✓	✓	5.4.4
	Interface friction	✓	✓	✓	5.4.7

[a]Estimates of M_R and ν are also required for shallow bedrock in new/reconstruction designs.
[b]From FWD testing for rehabilitation designs. For new/reconstruction designs, $k_{dynamic}$ is determined from Level 2 estimates of M_R.
[c]For Level 2, M_R may be estimated directly or determined from correlations with one of the following: *CBR*; *R*; a_i; *DCP*; or *PI* and *P200*.

Table 5-5. Thermo-hydraulic inputs required for the NCHRP 1-37A Design Guide.

Property	Description	Level 1	Level 2	Level 3	Section
	Groundwater depth	✓	✓	✓	5.5.2
Infiltration and Drainage					
	Amount of infiltration	✓	✓	✓	5.5.2
	Pavement cross slope	✓	✓	✓	5.5.2
	Drainage path length	✓	✓	✓	5.5.2
Physical Properties					
G_s	Specific gravity of solids	✓			5.3.2
$\gamma_{d\,max}$	Maximum dry unit weight	✓			5.3.2
w_{opt}	Optimum gravimetric water content	✓			5.3.2
PI	Plasticity Index		✓		5.3.2
D_{60}	Gradation coefficient		✓		5.3.2
P200	Percent passing 0.075 mm (No. 200 sieve)		✓		5.3.2
Hydraulic Properties					
a_f, b_f, c_f, h_r	Soil water characteristic curve parameters	✓			5.5.2
k_{sat}	Saturated hydraulic conductivity (permeability)	✓			5.5.2
PI	Plasticity Index		✓	✓	5.3.2
D_{60}	Gradation coefficient		✓	✓	5.3.2
P200	Percent passing 0.075 mm (No. 200 sieve)		✓	✓	5.3.2
Thermal Properties					
K	Dry thermal conductivity	✓			5.5.2
Q	Dry heat capacity	✓			5.5.2
	AASHTO soil class			✓	4.7.2

2. Physical properties

Physical properties provide the most basic description of unbound materials. The principal properties are specific gravity of solids, water content, unit weight (density), gradation characteristics, plasticity, classification and compaction characteristics. A sample of soil is a multiphase material consisting of soil grains, water and air. The weight and volume of a soil sample depends on the specific gravity of grains, the size of pores and amount of water.

Table 5-7. Terms in weight-volume relations (after Cheney and Chassie, 1993).

Property	Symbol	Units[1]	How obtained (AASHTO/ASTM)	Direct Applications
Moisture Content	w	D	By measurement (T 265/ D 4959)	Classification and weight-volume relations
Specific Gravity	G_s	D	By measurement (T 100/D 854)	Volume computations
Unit Weight	γ	FL^{-3}	By measurement or from weight-volume relations	Classification and pressure computations
Porosity	n	D	From weight-volume relations	Defines relative volume of solids to total volume of soil
Void Ratio	e	D	From weight-volume relations	Defines relative volume of voids to volume of solids

[1] F = Force or weight; L = Length; D = Dimensionless. Although by definition, moisture content is a dimensionless fraction (ratio of weight of water to weight of solids), it is commonly reported in percent by multiplying the fraction by 100.

Table 5-8. Unit weight-volume relationships.

Case	Relationship	Applicable Geomaterials
Soil Identities:	1. $G_s w = S e$ 2. Total Unit Weight: $\gamma_t = \dfrac{(1+w)}{(1+e)} G_s \gamma_w$	All types of soils & rocks
Limiting Unit Weight	Solid phase only: $w = e = 0$: $\gamma_{rock} = G_s \gamma_w$	Maximum expected value for solid silica is 27 kN/m^3
Dry Unit Weight	For $w = 0$ (all air in void space): $\gamma_d = G_s \gamma_w/(1+e)$	Use for clean sands and soils above groundwater table
Moist Unit Weight (Total Unit Weight)	Variable amounts of air & water: $\gamma_t = G_s \gamma_w (1+w)/(1+e)$ with $e = G_s w/S$	Partially-saturated soils above water table; depends on degree of saturation (S, as decimal).
Saturated Unit Weight	Set $S = 1$ (all voids with water): $\gamma_{sat} = \gamma_w (G_s+e)/(1+e)$	All soils below water table; Saturated clays & silts above water table with full capillarity.
Hierarchy:	$\gamma_d \geq \gamma_t \leq \gamma_{sat} < \gamma_{rock}$	Check on relative values

Note: $\gamma_w = 9.8$ kN/m^3 (62.4 pcf) for fresh water.

The specific gravity of soil and aggregate solids (G_S) is the ratio of weight of sample volume to the weight of equal volume of distilled water at a given temperature. It is used for calculation of soil volumetric properties (e.g. void ratio) and analysis of hydrometer test. Lab determination according to AASHTO.

Moisture content expresses the amount of water present in a quantity of soil. Gravimetric moisture content is the weight ratio of water present in a soil sample. Moisture content is used for calculation of volumetric properties (e.g. void ratio) and correlation with soil behavior. Lab determination with oven drying or field determination with nuclear gauge.

Unit weight (density) is the total weight of solid over volume and is used in calculation of *in-situ* stresses, correlation with soil behavior and compaction control. Lab determination with weight and volume measurements and field determination with a nuclear gauge or sand cone. Unit weight depends on moisture content.

Compaction (eq. 5.1-5.6) is one of the most important geotechnical properties for highway construction quantified as equivalent try unit weight of soil in a volume and depends on water content. Compaction increases elastic stiffness, decreases compressibility, increases strength, decreases hydraulic conductivity and void ratio and increases erosion resistance. Lab tests use AASHTO while in field tests unit weight and moisture are determined to check whether the material meets requirements. Relative compaction is the density ratio of test sample to maximum achievable density after compaction. The Specifications typically require a minimum level of compaction (e.g. 95%). Relative density is used for granular soils.

Gradation is the distribution of particle sizes within a soil determined with sieve analysis and is used for soil analysis and correlation with other parameters. Lab tests include mechanical sieve analysis (larger than 0.075 mm) for finer grains and hydrometer test for silt and finer clay materials (smaller than 0.075 mm). Sample preparation requires care as to obtain a representative result. The results are expressed as a fraction of sieve of the total sample.

Plasticity describes response of soil to moisture content with increased water content the soil transitions from rigid to soft. Clays are plastic while sands and gravels are non-plastic. Atterberg limits define transition: liquid limit LL (liquid and plastic), plastic limit PL (plastic and semi-solid) and shrinkage limit SL (semi-solid and solid). The Atterberg limits define basic indices: the plasticity index (PI=LL-PL) and liquidity index (LI=w-PL; w- natural water content in sample soil).

Two problematic soil conditions must be checked: swelling clays and collapsible silts. The amount of possible swelling depends on the clay amount, moisture content, relative density, compaction, permeability and overburden stress. Collapsible soils exhibit large moisture gradients. Both conditions can have severe detrimental impact on pavement performance.

Table 5-26. Other tests for aggregate quality and durability.

Property	Use	AASHTO Specification	ASTM Specification
Fine Aggregate Quality			
Sand Equivalent	Measure of the relative proportion of plastic fines and dust to sand size particles in material passing the No. 4 sieve	T 176	D 2419
Fine Aggregate angularity (also termed Uncompacted Air Voids)	Index property for fine aggregate internal friction in Superpave asphalt mix design method	T 304	C 1252
Coarse Aggregate Quality			
Coarse Aggregate Angularity	Index property for coarse aggregate internal friction in Superpave asphalt mix design method		D 5821
Flat, Elongated Particles	Index property for particle shape in Superpave asphalt mix design method		D 4791

General Aggregate Quality			
Absorption	Percentage of water absorbed into permeable voids	T 84/T 85	C 127/C 128
Particle Index	Index test for particle shape		D 3398
Los Angeles degradation	Measure of coarse aggregate resistance to degradation by abrasion and impact	T 96	C 131 or C 535
Soundness	Measure of aggregate resistance to weathering in concrete and other applications	T 104	C 88
Durability	Index of aggregate durability	T 210	D 3744
Expansion	Index of aggregate suitability		D 4792
Deleterious Materials	Describes presence of contaminants like shale, clay lumps, wood, and organic material	T 112	C 142

3. Mechanical properties

Stiffness is the most important mechanical characteristic of unbound materials and the relative stiffness of layers determines the distribution of stresses and strains within the pavement system. The preferred method for characterizing stiffness of unbound pavement materials is the resilient modulus M_R, defined as the unloading modulus at cyclic loading. In parallel. the California bearing ratio (CBR) and R-index are still widely used as inputs for empirical pavement design and can be correlated with M_R.

California bearing ratio (CBR) is an indirect measure of soil strength based on resistance to penetration by a piston. CBR is defined by the ration of load required to cause a certain penetration depth with compacted soils to the standard load required to obtain same depth with standard sampled of crushed rock. It is a relatively easy and inexpensive method with long history in pavement design. CBR is a direct input into empirical design and can be correlated with M_R. It is mostly determined in the lab with accuracy depending on how representative the tested samples are.

Stabilometer (R-value) (eq. 5.7) is a measure of resistance to deformation and expressed as a function of the induced lateral pressure to applied vertical pressure. It is a direct input to some empirical methods and correlated with CBR and M_R.

Elastic (resilient) modulus (eq. 5.8-5.12) was defined when it was found that the initial modulus from single unconfined compression loading differs considerably from M_R at repeated cyclic loading. M_R is used to characterize the elastic modulus of unbound materials, subgrade stiffness and determination of structural layer coefficients in flexible pavements. It is defined as the ratio of applied cyclic stress to the elastic strain after many cycles of loading ($M_R = \Delta\sigma/\Delta\varepsilon_a$) and thus a direct measure of stiffness. It is the key soil parameter and a direct input to both AASHTO and NCHRP 1-37A design guides. Most common method (also table 5-31) for determination of M_R is the triaxial test where the sample is initially hydrostatically compressed and then exposed to constant cyclic loading. The advantages of the triaxial test are: it is conducted with simple apparatus, the stress conditions are similar in magnitude to those occurring at wheel loading, it allows for control over drainage and pore pressure of samples, easy measurement of strain and sample preparation and availability of test protocols. The lab measured M_R is stress dependent: M_R increases with stress for coarse grained materials while M_R decreases with stress for fine grained materials. Seasonal variations of unbound materials are taken into account by measuring M_R after a certain time period (e.g. 1 month) throughout the year. M_R can also be estimated *in-situ* from back-calculations from FWD measurements. The hierarchical levels of

M_R depending on the construction type and correlations with other material properties are shown in tables.

Table 5-33. Hierarchical input levels for unbound material stiffness in the NCHRP 1-37A Design Guide (NCHRP 1-37A, 2004).

Project Type	Level 1	Level 2	*Level 3*
Flexible Pavements			
New/reconstruction	Laboratory-measured M_R with stress dependence—Eq. (5.10)	M_R correlations with other properties (Table 5-34)	Default M_R based on soil type (Table 5-35)
Rehabilitation	Backcalculated M_R from FWD deflections	M_R correlations with other properties (Table 5-34)	Default M_R based on soil type (Table 5-35)
Rigid Pavements			
New/reconstruction	Not available	M_R correlations with other properties (Table 5-34)	Default M_R based on soil type (Table 5-35)
Rehabilitation	Backcalculated modulus of subgrade reaction (k) from FWD deflections (see Section 5.4.6)	M_R correlations with other properties (Table 5-34)	Default M_R based on soil type (Table 5-35)

Table 5-34. Correlations between resilient modulus and various material strength and index properties (NCHRP 1-37A, 2004).

Strength/Index Property	Model[a]	Comments	Test Standard
California Bearing Ratio[b]	M_R (psi) = $2555(CBR)^{0.64}$ M_R (MPa) = $17.6(CBR)^{0.64}$	CBR = California Bearing Ratio (%)	AASHTO T193—The California Bearing Ratio
Stabilometer R-value	M_R (psi) = $1155 + 555R$ M_R (MPa) = $8.0 + 3.8R$	R = R-value	AASHTO T190—Resistance R-Value and Expansion Pressure of Compacted Soils
AASHTO layer coefficient	M_R (psi) = $30{,}000\ (a_i/0.14)^3$ M_R (MPa) = $207\ (a_i/0.14)^3$	a_i = AASHTO layer coefficient	AASHTO Guide for the Design of Pavement Structures (1993)
Plasticity index and gradation	$CBR = \frac{75}{1+0.728(wPI)}$	$wPI = P200*PI$ $P200$ = % passing No. 200 sieve size PI = plasticity index (%)	AASHTO T27—Sieve Analysis of Coarse and Fine Aggregates AASHTO T90—Determining the Plastic Limit and Plasticity Index of Soils
Dynamic Cone Penetration[c]	$CBR = 292/(DCP^{1.12})$	CBR = California Bearing Ratio (%) DCP =Penetration index, in./blow	ASTM D6951—Standard Test Method for Use of the Dynamic Cone Penetrometer in Shallow Pavement Applications

[a]Correlations should be applied to similar conditions – *i.e.*, CBR measured at optimum moisture and density vs. soaked conditions correlates to M_R at corresponding moisture and density conditions.
[b]NCHRP 1-37A strongly recommends against use of the older Heukelom and Klomp (1962) correlation Eq. (5.13) between M_R and CBR specified in the 1993 AASHTO Design Guide.
[c]Estimates of CBR are used to estimate M_R.

The elastic modulus of fractured PPC slabs needs to be considered in rehabilitation design and typically level 1 design inputs are sufficient. Shallow bedrock under a pavement alignment can have significant impact on mechanical response of the pavement system and is therefore important to be taken into account in mechanistic-empirical designs.

Poisson's ratio is defined as the ratio of lateral strain to axial strain due to axial loading. It used as input into mechanistic-empirical design and for estimation of *in-situ* lateral stresses. The influence of Poisson's ratio on pavement response is typically small and assumed values give satisfactory results so lab testing is rarely needed.

Structural layer coefficients for base and subbase layers are purely empirical and characterize material quality. This parameters were determined for a range of materials in AASHTO guide and they are not fundamental engineering properties. In addition to material quality, the position of the layer in the pavement system is important. The structural layer coefficients (a_1, a_2 and a_3) for new construction/reconstruction projects are empirically (eq. 5.16-5.18) correlated with the resilient modulus of base/subbase layers (tables 5-41 to 5-43 and figures 5-19 to 5-22). During rehabilitation projects the values assigned to these coefficients should be less (table 5-44).

Modulus of subgrade reaction (eq. 5.19) is a parameter describing the stiffness of the foundation soil in mechanistic solutions however it also depends on the stiffness of the slab/footing and is therefore not a fundamental engineering property. It is a direct input to both the AASHTO and NCHRP 1-37A guides for new construction/reconstruction. AASHTO guide suggests the following steps: identify of subgrade/subbase type and properties; determine M_R; determine subbase E_{SB}; determine k values from figures 5-24 and 5-25; determine the seasonal average k and finally correct k for erosion. NCHRP 1-37A guide characterizes for all subgrade and unbound pavement layers with the M_R and establishes the following steps: assign initial M_R values for layers; compute stresses in subgrade/subbase and simulate FWD load with elastic theory; Adjust M_R with stresses; with elastic theory update the model to calculate PCC surface deflection and determine k.

Interface friction between the slab and the underlying material is characterized in the AASHTO guide by the frictional factor F (related to frictional coefficient) and in the NCHRP 1-37A guide by the interface coefficient varying between 0 (full slip) to 1 (fully bonded).

Permanent deformation characteristics of unbound materials are used in NCHRP 1-37A in empirical distress models for rutting of flexible pavements. Determination is with triaxial repeated cyclic loading and equations 5.27- 5.35.

Coefficient of lateral earth pressure (eq. 5.36-5.39) is defined as the ratio of horizontal to vertical *in-situ* effective stress.

4. Thermo-hydraulic properties

Thermo-hydraulic parameters characterize the moisture and temperature conditions in the pavement system and material properties. Temperature greatly affects asphalt stiffness and temperature gradients can result in curling and stresses. Moisture content affects the stiffness and strength of unbound materials. Combined effect (e.g. freeze/thaw cycles) can be detrimental.

Inputs to the AASHTO guide are empirical and include drainage (included in SN), swelling and frost heave parameters (included in ΔPSI). The improved pavement performance due to drainage is included in design in the structural number (eq. 5.40). Swelling and frost heave both include three empirical parameters: rate constant (fig. 5-34 and 5-36), probability and potential vertical rise/max serviceability loss (fig. 5-33 and 5-35) for swelling and frost heave, respectively.

The NCHRP 1-37A guide requires inputs: groundwater table depth, infiltration and drainage properties, physical properties, soil water characteristic curve, hydraulic and thermal conductivity and heat capacity. *The groundwater depth* plays important role for predicting seasonal moisture content of unbound layers and thus M_R values. It can be acquired from borings. *Infiltration and drainage* are characterized by infiltration amount (climate parameters, drainage features and pavement shoulder type), pavement cross slope (slope of surface perpendicular to traffic) and drainage path length (resultant of cross and longitudinal slopes). Infiltration should be kept at a minimum. *Physical properties* include specific gravity for solids, maximum dry unit weight and optimum gravimetric moisture content (table 5-52). *Soil water characteristic curve* (fig. 5-37) determines the relationship between water content and matric suction for a material(eq. 5.51- 5.53). The curve is typically obtained from other easily measurable parameters (table 5-53). *Hydraulic conductivity* describes the ability of the material to conduct water and is defined as the quantity of fluid flow through unit area at unit pressure. It is used to estimate drainage characteristics. The models require unsaturated conductivity but in practice it is estimated by measuring saturated conductivity (eq. 5.66 and table 5-54 and 5-55). *Thermal conductivity* is the material's ability to transport heat (table 5-58). *Heat capacity* of the amount of heat required to raise the temperature on a unit of material by one degree (table 5-58).

5. Environment/climate inputs

The AASHTO guide requires only three empirical parameters: estimated seasonal variation, the % of time that unbound materials are exposed to moisture near saturation and the qualitative description of moisture supply for expansive soils. The NCHRP 1-37A requires parameters: climate, groundwater depth and surface absorptivity. The climate is taken into account with variables such as air temperature, precipitation, wind speed, percentage sunshine and relative humidity. Groundwater depth variation can significantly affect season values for modulus and should be determined accurately. Absorptivity is the fraction of solar energy absorbed by the surface of the pavement.

6. Development of design inputs

The gaps in the knowledge for pavement design (especially material variation) can cause significant uncertainties with any design. Some general guidelines have been established to reduce errors.

- Careful analysis of the available data with proper statistical tools to identify and exclude outliers (typical ranges for variables in table 5-60). However, the outliers may also anomalies in subsurface properties.
- Examine spatial (and temporal) trends in the data.
- Check for order of magnitude errors and trends in the data.
- Examine internal consistencies of data (e.g. volume vs weight relationships).
- Use correlations among data (e.g. correlation of CBR with M_R in case of few M_R test locations).
- Be clear what design value input is required.
- Most importantly: determine the sensitivity of the design model to input parameters. This can greatly improve the accuracy (by determining the more sensitive inputs more carefully) and reduce cost (e.g. fewer test locations for insensitive parameters).

Chapter 6 Pavement structural desing and performance

The aim of this chapter is to provide qualitatively evaluate how much different geotechnical properties influence the performance of the pavement. A baseline new construction project is used as comparison for conditions such as weak/soft subgrade conditions, subgrade stabilization, low quality base/subbase material, drainage and water conditions and shallow bedrock conditions. Both the AASHTO and the NCHRP 1-37A design guides are used for flexible and rigid pavements. The outputs of the simple AASHTO design are layer thicknesses that match the design criteria. The NCHRP 1-37A guide evaluates the pavement performance in time through calculations with models. Typically, several designs meet the design requirements and while crude initial construction cost can be used as estimate the final decision should also include lifetime servicing costs.

Baseline designs

The design is based on a new construction with simple pavement structure: hot mix asphalt (HMA) over stone graded aggregate base (GAB) over subgrade (SG) for flexible pavements and jointed plain concrete pavement (JPCP) over GAB over SG for rigid pavement. Base material is expected to be non-erodable and excellent, SG is non-expansive, good drainage, simple traffic (constant over design period) and benign climate factors (e.g. no frost heave/thaw) and no seasonal variation. Typical materials costs are in table 6-1.

Table 6-1. Typical in-place unit material costs for use in example design problems (MDSHA, 2002).

Material	Reasonable Range	Typical Unit Price	Typical Unit Price
Hot mix asphalt concrete (12.5 mm PG 64-22)	\$30-\$50/ton	\$36/ton	\$14,250/lane-mi-in
Portland cement concrete (PCC) without steel	\$110-\$180/cy	\$144/cy	\$28,200/lane-mi-in
Graded aggregate base	\$24-\$60/cy	\$42/cy	\$8,200/lane-mi-in

The AASHTO guide gives for flexible pavements the required overall structural number as SN= 4.61, for asphalt SN_1= 2.35 resulting in thickness of D_1= 5.3 inches (135 mm). Remaining structural number for granular base layer SN_2= 2.28 and layer thickness of D_2= 12.7 inches (323 mm). Initial cost would be lowered by replacing 2.4 inches of GAB with 1 inch of AC (due to the ratio of layer coefficients a_1/a_2 being greater than unit cost of materials) resulting in 5.3 inches of AC and 12.7 inches of GAB for a predicted lifetime of 15 years.

For rigid pavement the design includes: assume a 6 inch granular base thickness; determine the composite modulus of subgrade reaction k_∞ (combined stiffness for subbase and subgrade); correct k_∞ for loss of support to determine the design modulus od subgrade reaction k_{eff}; determine the required slab thickness (10 inches (254 mm) in this case). The final design is 10 inches of PCC slab over 6 inches of GAB.

NCHRP 1-37A design for flexible pavements also consists of three layers: AC, GAB and SG but the models require substantially more input parameters (table 6-4 to 6-7) than the AASHTO guide. The design limit for total rutting is an input parameter determined by local policies and the model output is compared to this parameter after evaluating trial pavement section designs (figure 6-1). The values from the AASHTO design keep the rutting values below the limit at 0.65 inches and thus meet the design requirements.

The rigid pavement structure is also the same three layer JPCP construction (PPC+GAB+SG). As with flexible pavements the total faulting is evaluated (0.117 inches at 25 years) with ASHTO outputs as trial section and these meet the design requirements.

Soft subgrade

The soft subgrade case has everything identical to baseline design except for the M_R or subgrade (only 6000psi). To meet the same design criteria this expectedly increases the pavement thickness or increased quality.

The AASHTO guide for flexible pavements now requires overall structural number as SN= 5.76, for granular base layer SN_2= 2.28 (layer thickness of D_2= 12.7 inches or 323 mm) and for asphalt SN_1= 3.48 resulting in thickness of D_1= 7.9 inches (203 mm) i.e. an increase of 50% AC thickness and 20% increase in initial cost. In case of rigid pavements, the PCC slab thickness is relatively insensitive to foundation stiffness increasing only by 0.1 inches to 10.5 inches of PPC and 6 inches of GAB. A common concern is that soft conditions make construction conditions difficult.

NCHRP 1-37A design for flexible pavement modified parameters are in table 6-11. Rutting is again the primary faulting mechanism and the final design meeting the design criteria is identical to that provided by AASHTO (7.9 inches of AC and 12.7 inches of GAB). For rigid pavements the slab thickness is increased by 5% to 10.9 inches of PCC and 6 inches of GAB.

The soft subgrade conditions would in this case increase cost of flexible pavements by about 20% according to both designs compared to the baseline case. The cost of rigid pavements would increase by 1% and 8% for the AASHTO and NCHRP 1-37A designs, respectively.

Subgrade stabilization

Lime stabilization is a common technique to improve weak/soft subgrades under flexible pavements (rigid pavements only slightly affected by soft/weak subgrade). Lime stabilization increases stiffness with main parameters being lime content and stabilized layer thickness (figure 6-9). Whether lime stabilization economical is dependent on the cost of lime in the construction area.

Low quality base/subbase

This case has a lower subbase material quality compared to baseline design which results in thicker design pavement thickness for same level of performance.

In case of **the AASHTO guide** for flexible pavements the subbase M_R and layer coefficient are reduced by 35% and also the drainage coefficient is reduced by 30% resulting in a decreased SN (by 86%) in same service life. The SN provided by granular base layer (thickness 12.7 inches) is 1.07 and the design requirement of SN= 4.6 dictates that AC should provide additional SN= 3.54 (thickness 8 inches or 203 mm). In case of rigid pavements MR is decreased, so is the drainage coefficient while the loss of support coefficient is increased resulting n 50% decrease in initial service life. To achieve design life requirement the PCC thickness must be increased to 11.5 inches (292 mm) with 6 inches of GAB.

According **NCHRP 1-37A** design for flexible pavements low quality base/subbase reduces design life by 15% (table 6-10) with rutting being the primary distress factor. Design limit for rutting of 0.65 inches results in the final design of 5.8 inches AC over 12.7 inches GAB which is significantly thinner than suggested by AASHTO for the same conditions. For rigid pavements (table 6-14) the design life decrease is only 6% and final design is 10.7 inches of PPC over 6 inches of GAB.

Expectedly the low quality base/subbase resulted in thicker pavement cross section. Initial construction costs of flexible pavements increased by 21% and 4% for AASHTO and NCHRP 1-37A design, respectively. For rigid pavements the initial cost increase was 9% and 2.5%.

Poor drainage

In the poor drainage case material properties remain the same as in the baseline case, however drainage is very poor resulting in: decrease of stiffness (and strength) of the granular base and subgrade layers because of higher average moisture content; increase in moisture related distresses.

In case of **the AASHTO guide** for flexible pavements the seasonally averaged M_R and layer coefficients for granular base and subgrade are reduced by about 40% due to moisture, also the drainage coefficient is reduced and moisture conditions approach saturation 5-25% of time. Design life decreases by 89%. Overall required SN is till 5.76 while the SN provided by base is reduced to 0.91 correspondingly increasing the required SN from AC to 4.76 and AC layer thickness of 10.8 inches (100% increase compared to baseline). For rigid pavements M_R and drainage coefficients for base/subgrade are reduced while the loss of support increases and the corresponding design life loss is 68%. The design that meets design requirements is 12.3 inches of PCC and 6 inches of GAB. The increase of slab thickness is primarily due to poor drainage.

According **NCHRP 1-37A** design does not allow to model inefficient drainage for flexible pavements. For rigid pavements poor drainage results in 6% in initial service life and the final design is 10.7 inches of PCC on 6 inches of GAB which is only an increase of 2.5% compared to baseline.

Poor drainage conditions lead to an increase of 44% in initial costs in case of the AASHTO design. For rigid pavements the increase is 16% and 2.5% according to AASHTO and the NCHRP 1-37A guides, respectively.

Shallow bedrock

The AASHTO guide for flexible pavements does not include shallow bedrock into design. For rigid pavements the shallow bedrock increases the design modulus of subgrade reaction reducing slightly the final design values of 10.3 inches of PCC over 6 inches of GAB.

In the **NCHRP 1-37A** for flexible pavements depth to bedrock is an explicit design input (table 6-4) and a depth of 1220 mm the design life is increased to 120% and design value can be reduced to 3 inches of AC over 12.7 inches of GAB. In case of rigid pavements depth of 1220 mm to bedrock reduces joint faulting by 75% and slab thickness is reduced to below 7 inches (not modellable by program) at 6 inches of GAB.

Concluding comments

The illustrated design scenarios indicate the sensitivity of various design to input parameters. The ultimate measure for comparing designs is the cost and some of the key points are: poor drainage is by far the most detrimental geotechnical factor for flexible pavements; soft/weak subgrade is the second most detrimental geotechnical factor for flexible pavements but can be mitigated with stabilization; lower quality materials increase layer thicknesses but the overall cost might not change; rigid pavement design is much less sensitive to geotechnical parameters than flexible pavements.

Chapter 7 Desing details and construction conditions requiring special attention

Pavements system could additionally contain drainage systems and/or filter materials (e.g. geotextiles).

Subsurface water and drainage requirements

The negative effect of water that enters the pavement system through various sources is well known and together with heavy traffic or freeze/thaw cycles can be detrimental to performance. Sources of moisture from below the pavement are capillary faction and high groundwater table which can in some areas be higher than the pavement system itself. In that case drainage systems to block the water or rapidly remove it from the pavement system are required. The most of excess water enters the pavement system via infiltration through cracks, joints and shoulder edges (about 40% of rainfall enters the pavement system) and the problems worsen with time i.e. aging of the pavement.

Moisture accelerates damage to the pavements e.g. materials become unstable at saturation and dynamic loading causes high water pressure, additionally freezing expansion substantially damages the pavement. The deteriorating effects of water saturation include: water in AC surface leads to damage, modulus reduction (up to 30%) and loss of tensile strength; unbound base/subbase and fine-grained soils loose stiffness and modulus (up to 50%); increased erosion. Saturation effect on design life is illustrated on figure 7-1.

There are three approaches to keep the susceptible base/subbase/subgrade materials from becoming saturated: prevent moisture from entering pavement system; use materials and design features which are insensitive to moisture; quickly remove moisture which enter the pavement system. No single approach can alone completely eliminate water and typically several methods must be combined (e.g. drainage features with preventative measures or materials) this however increases both initial and maintenance costs.

Drainage in pavement systems can be accomplished either by vertically draining the water into subgrade or laterally with a drainage layer to a collector (or a combination of both). Table 7-3 assesses the need for drainage. The drainage quality is defined by the time-to-drain which is the time required after a significant rainfall event (or other event) to drain from a saturated state to a specific saturation (typically 50%). Effect of drainage is included in both the AASHTO and NCHRP 1-37A guides either as drainage and modified layer coefficients or interaction between climatic and structural factors. In high rainfall areas excellent drainage can reduce the thickness of flexible pavement base by a factor of 2 (or increase design life by a factor of 2). On the other hand, achieving poor drainage is relatively simple if the subgrade is not free draining and dense graded subbase is used without drainage features. In order to achieve good or excellent drainage a more permeable open-graded base and/or subbase should be used and tied to a subsurface drainage system.

Subsurface drainage systems are typically classified according to location and geometry into: longitudinal edgedrains, transverse and horizontal drains, permeable bases, deep drains or underdrains and interceptor drains. These can control several sources of moisture and perform several functions. Drainable pavement systems generally consist of the following design features: a full-width permeable base under AC or PCC; a separator layer under permeable base to avoid segregation; longitudinal edgedrains. *Daylighted bases* consist of permeable base under the AC or PCC layer that extends the full-width of the pavement into the ditches. *Longitudinal edgedrains* consist of a drainage system that runs parallel to the traffic lane and include both pipe and aggregate drains. The most commonly used edgedrain is the perforated pipe with a diameter of 100-150 mm. Edgedrain systems can be very effective when used together with other features e.g. permeable bases. Edgedrains can be constructed with geocomposites (difficult to maintain) and aggregate trenches (low hydraulic capacity). The most critical item for edgedrains is the grade of invert and the required slope depends on the required hydraulic capacity. In most cases rigorous maintenance is expected. *Permeable base* is designed to rapidly move surface infiltrated water to the side ditches and should contain no fines. Permeability values in excess of 300m/day and thickness of 100 mm are suggested. *Dense-graded stabilized base with permeable shoulders* consists of nonerodable typically lean concrete base or asphalt treated base under traffic lanes and permeable base under the shoulder. *Horizontal geocomposite base* has been used above dense graded base or as drainage layer between beneath full

AC or between cracked PCC and new layer of AC during rehabilitation and tied into an edgedrain system. Such systems have been found to have excellent drainage. *Separator layers* are essential with pavements with permeable bases by preventing contamination with fines into the layer. Various materials have been used as separator layers: geotextiles, dense-grade aggregates, asphalt chip seals, dense-graded AC and cement treated granular material. Geotextiles are beneficial when additional subgrade support is not necessary and in case of sensitive subgrades, also allowing for more open-graded, free-draining subbase.

The effect of drainage (and moisture damage) on pavement performance is substantial, especially because traffic loadings have increased over the past decades i.e. a pavement may be adequately drained for a level of traffic but increased traffic loadings increase the damage caused by moisture. However, the benefits of drainage are negated (and moisture damage occurs) in cases: inadequate design of permeable bases and separator layers and edgdrains; lack of construction quality control; lack of maintenance; possible settlement of PCC slab over untreated aggregate permeable bases. Permeable bases must be constructed of durable, crushed aggregate for good stability and must have separator layers that prevent entering of fines to the permeable base. Permeable bases must be connected to adequate edgedrains and maintained regularly.

Design of pavement drainage consists of determining: hydraulic requirements of the permeable layer; edgedrain pipe size and outlet spacing; gradation requirements for aggregate separator layers or sufficient properties (strength, endurance, opening size, permeability) of geotextile separators; edgedrain properties for aggregates of geotextiles. There are two approaches for hydraulic requirements: time-to-drain and steady state flow. Typically, the time-to-drain approach (time to drain 50% of free water following a moisture event that caused saturation) is used (eq. 7.1) and is affected by the time factor (thickness, slope and length of permeable layer; eq. 7.2) and drainage layer properties (effective porosity and coefficient of permeability; eq. 7.3). Time-to-drain decreases exponentially with permeability and slope values and increases linearly with length and effective porosity, while thickness has very little effect. *Edgedrain pipe size and outlet spacing* are recommended to be 100 mm pipe diameter and 75 m. According to the flow capacity method of circular pipes (eq. 7.4) would result in capacity of drainage system of 504m^3/day which is more than adequate but the 100 mm pipe is recommended for maintenance. According to the infiltration method (eq. 7.5-7.6) a design rainfall and infiltration ratio are selected and comparing the discharge rate of the system with the flow capacity of the drainage layer.

Separator layer consists of aggregate material or a geotextile layer to prevent mixing of base/subbase layer with the permeable layer. It may be beneficial to use a low permeable layer as separator to deflect water from entering lower layers. If a dense graded aggregate is used it should be a hard and durable material: two crushed faces and consist of 98% crushed stone; abrasion wear below 50%; soundness loss below 12%; gradation allowing for max permeability of 5m^3/day; material passing the 425 mm sieve shall be non-plastic. *A geotextile separator layer* must prevent intermixing and satisfy filtration criteria. Both woven and non-woven geotextiles have been used. Geotextiles can also be used as filter for edgedrains and prevent the soil from washing up and clogging over time. A geotextile design with proper performance should include: determine gradation of filtered material and % finer than 0.075 mm sieve; determine permeability of base/subbase; apply design criteria to determine apparent open size, permeability and permittivity; geotextile must be strong enough to survive construction and installation.

Base layers: requirements, stabilization, reinforcement

The function of base course varies depending on the pavement type. Under rigid pavements it: provides uniform and stable support; minimizes frost action damage; provides drainage; prevent volume change of subgrade; prevent pumping of fine grained soils at joints; increase structural capacity and expedite construction. Under flexible pavements base course structurally improves the load supporting capacity of pavement by added stiffness and resistance to fatigue. The base course should be: either free draining or highly resistant to erosive action of water; should not contain material finer than 0.075 mm; non-frost susceptible; may be stabilized with asphalt or concrete; well-graded and resist deformation due to loading. The material for base course should be with at least two fractured sides and consist of crushed stone; abrasion wear below 45%; soundness loss below 12%; gradation should allow for free movement of water.

Erodibility of bases is the loss of material due to hydraulic action at the joints or edges of pavement. Erodibility is related to durability of base and the potential to break down due to dynamic traffic loads, climatic conditions, environmental and water action. Traffic level is a critical factor for base/subbase erosion. Another problem is the erosion of fine-grained soils under a stabilized base. This can be overcome by placing a dense-graded subgrade or a geotextile separator layer between the base coarse and compacted subgrade. Tables 7-6 to 7-8 provide material classifications for the NCHRP 1-37A guide.

Bound bases are bases with a sufficient quantity of stabilizing agent to increase tensile strength to a required level.

Modified (or treated) bases are materials with addition of less than 5% asphalt or cement to iprove stability for construction and behave structurally as unbound granular materials. Most often permeable layers tend to rut under construction and require modification (tables 7-9 and 7-19). Density control of the modified material is important and while modification is often used to reduce the layer thickness it is not advised to reduce thickness below 150 mm.

Base reinforcement is the use of geosynthetics strengthen base layer in flexible pavements (table 7-11). Main function is to provide lateral confinement of the aggregate layer. Reinforced sections have shown a factor of 10 improvement in design life/performance. The technology is not fully developed and should be carefully analyzed for project based on the increase in structural parameters (and design life/pavement performance) and the cost to employ the reinforcement.

Compaction

Compaction of subgrade and unbound base/subbase materials a fundamental method in pavement construction to increase stiffness and strength and decrease the permeability and erosion of materials. Intent of compaction is to maximize soil strength by proper adjustment of moisture and densification. The higher density requirements are typically employed for at least 0.6 m of topmost subgrade.

Compaction theory dates back to the 1930s and impact compaction with a falling hammer is the standard lab test. The resulting soil strength depends on the density (compaction energy), moisture content and the soil type (figures 7-11 and 7-12). Compaction specifications are based on achieving minimum dry weight expressed as relative compaction (eq. 7.5). Clays are especially sensitive to moisture content (figure 7-13).

The principal effect of compaction on soil properties is an increase of density; increase of strength with increasing compaction energy, this however depends also on the moisture content (saturation greatly reduces strength) and soils type (figure 7.14); stiffness increases with compaction energy dry of optimum moisture and is independent of compaction energy wet of optimum (figure 7-15); permeability at constant compaction energy decreases with increasing moisture (minimum at optimum); swelling/shrinkage of clays is greater when compacted dry/wet of optimum, respectively (figure 7-16).

Subgrade conditions requiring special design attention

The variables such as soil type, mineralogy, geology, groundwater depth and flow properties can change over the length of the construction project and should be evaluated. Often such variables as collapsible/expansive soils or frost heave/thaw cycles are geographically regional and should be taken into account during design. A complex subsurface exploration program will certainly help to identify such variables and design approaches can mitigate/mimeses their effect, but such measures are often not available.

Problematic soil types are listed on pages 7-49 to 7-52. General approaches for dealing with problematic soils include: improved drainage; removal and replacement with borrow; mechanical stabilization with granular layers or geosynthetics or admixture; lightweight fill; soil encapsulation.

Compressible soils (very low density, saturated soils, slits, clays, wind-blown deposits) are susceptible to large settlements and deformations with time that can detrimentally affect the pavement performance unless properly treated. The treatment method depends on the soil type, depth and required compaction strength requirement for pavement: remove and replace with borrow or reprocess for optimal moisture/compaction; mechanical stabilization with geosynthetics; draining of saturated soils; consolidation with large fills; use of piled embankments or lightweight fill (e.g. geofoam).

Collapsible soils (very low density, alluvium, wind-blown deposits) can also lead to localized pavement failure. Their natural structure has typically been cemented with clay or other binders and upon saturation this structure breaks down. Special measures for treatment include: ponding water over the region; infiltration wells; compaction conventional or dynamic; excavation and replacement.

Swelling soils are susceptible to volume change (swell/shrink) with seasonal fluctuations on moisture content which depends on the soil type. This is typical of clay type soils. The extent of volume change varies for different clays and therefore the clay type should be identified during the subsurface exploration program. Treatment can be done in several ways: think layers of clay near the surface can be removed and replaced with borrow; extend pavement width to reduce change by reducing infiltration; partial/full encapsulation; scarify/stabilize/recompact the upper part of the clay subgrade with e.g. lime or cement; in case of over-consolidated clays perform the cuts and allow subsurface soils to rebound before construction.

Subsurface water (saturated soil strata, depth to groundwater, subsurface water flow) can severely reduce strength/stiffness of base/subbase if allowed to saturate with moisture. Cut and fill borders are particularly sensitive to seasonal moisture changes. Treatment techniques include: drying or mechanical stabilization of saturated soils near surface; remove and replace with borrow; increase thickness between pavement structure and saturated soils/groundwater table; use subgrade drains.

Frost-susceptible soils cause differential heaving, cracking and loss of bearing capacity due to seasonal temperature variations. The magnitude of these effects depends on soils type, moisture and climate. Frost-heave is caused by crystallization of ice lenses in voids of soils due to three simultaneous effects: frost-susceptible soils, water availability and sub-freezing temperatures in soil. The second consequence of frost action is thaw weakening and reduced bearing capacity. Freeze-thaw cycles have the worst effect. Conditions associated with high frost action possibility include: a water table within 3m of pavement structure; observed frost heave in area; inorganic soils containing more than 3% of grains finer than 0.02mm; potential for high groundwater table or ponding of surface water. Alternatives for improving frost-susceptible soils: remove and replace with borrow; place a non-susceptible borrow into subgrade; remove isolated pockets to avoid gradients; stabilize by cementation/drainage/altering soil freezing point; increase pavement structural layer thickness to account for strength loss. Pavement design includes three approaches: non-susceptible materials for entire depth of frost; permit frost penetration into pavement which does not reduce strength below requirement; allow frost penetration but compensate for weak thaw periods.

Subgrade improvement and strengthening

Proper identification and treatment of problematic soil conditions and preparation of foundation are essential to ensure long-lasting pavement performance. Subgrade stabilization provides an alternative to excavation and replacement of problem soils with the aim to achieve uniformity of the foundation in texture, density and moisture in the upper part of the subgrade.

Objective of soil stabilization is to make the subgrade soil suitable to act as pavement foundation by improving its strength/stiffness, reduce moisture susceptibility and restrict volume changes. Commonly used stabilization methods in table 7-13. Mechanical stabilization with thick gravel or granular layers is suitable for soft/wet subgrades. Geotextiles and geogrids together with quality aggregates can be used to reduce the extent of stress on the subgrade and avoid contamination of base/subbase layers with fines from the subgrade and maintaining the base thickness in time. Stabilization with admixtures of lime, cement and asphalt are used to prevent frost action and improve strength of subgrade. Lime is used for clays reducing the plasticity index (PI) and reducing frost susceptibility.

Characteristics of stabilized layers are improved (strength/stiffness) during stabilization but these values are typically not included in pavement design. Alternately, geosynthetics with unbound aggregates can be used as design inputs providing a surface reaction value. The improvements of subgrade from admixtures fill voids in the soils reducing in reduced permeability and reduce future volume changes. Increased strength can be a result of soil particles binding together due to the binding agent (e.g. cement) and also from reduced voids in the soil. Admixtures should be tested in the lab before construction. The zone selection for improving is dependent on traffic loads, drainage, underlying soil, depth of soil.

Thick granular layers provide benefits such as increased load-bearing capacity, frost protection and drainage and as construction support. The strategy is to achieve desired pavement performance through improved foundation characteristics. It can be used in areas with available quality aggregates instead of subgrade stabilization or increasing pavement layer thickness. It can fulfill several objectives: increase supporting capacity of weak/soft subgrades; provide uniform support over varying soil conditions; provide minimum support for construction activities; reduce season effects of moisture and temperature; promote surface runoff; improve drainage and removal of water from pavement; increase elevation of pavement in areas with high groundwater table; provide frost protection; reduce subgrade rutting; reduce pumping and erosion under PCC; meet elevation requirements. The most important properties of thick granular layers are thickness and material quality (strength/stiffness) as characterized e.g. by CBR. Aggregate gradation and shape are other important properties. Typically, embankment materials are dense-graded with particle diameter in range of 100-200mm. Generally, a granular layer of 0.5- 1.5m are used with thinner layers (ca 0.3m are used to support construction) and layer thickness above 2 m results in diminishing returns. In pavement design a total subgrade reaction value is assigned to the soils which is modified by a granular thick layer. The new value for subgrade reaction is a combination of the soil and aggregate values and should be determined experimentally (e,g, FWD). Aggregate layer thickness below 1m has negligible effect on subgrade reaction.

Geosynthetics are geotextiles and geogrids that consist of polymeric materials. Most typical uses are as reinforcement of embankments, separation of base/subbase materials from subgrade, creating barriers for water flow and improving drainage. Reinforcement is provided by laterally renstraining the base or subbase materials and improving the bearing capacity. Geotextiles serve best as filtration and separators and drainage layers while geogrids are better for reinforcement. Geosynthetic with appropriate aggregate provide excellent stabilization for weak/soft subgrades with CBR of less than 3 but do not provide frost protection nor expansive soils. Separation/filtration function can greatly improve the pavement performance as a relatively low cost by simply keeping pavement layer separate and retaining their structural strength/stiffness. The design using geosynthetics uses design-by-function approach consisting of several steps: identify subgrade conditions; determine whether a geosynthetic is required (table 7-15); design pavement without geosynthetic; determine need for addition aggregate layers and reduce its thickness 50% with use of geosynthetics and thickness of aggregate for construction platform; select the greater thickness; check infiltration criteria; determine geotextile survival and placement criteria.

Admixture stabilization variation possibilities are shown in table 7-16 including admixtures of cement, asphalt and lime. *Lime treatment* contains adding 1-3% of hydrated lime to enhance drying and compaction. Lime can also be used to treat swelling soils characterized by plasticity (indicator of swell potential). Depth to use lime stabilization is 0.6-1m.

Lime stabilization is used to change the chemical composition and increase strength of fine-grained soils. It can be used as drying during compaction, reduces swelling and volume changes and improve workability. Downside is that it can change some soils more frost-susceptible possibly due to inadequate curing. Most often used are hydrated lime, quick lime and high-calcium limes. For stabilization of clays lime content should be 3-8% of dry weight of soil (optimum % determined in lab). PI of about 10 is used as the lower limit for suitability for lime stabilization. Lime stabilization of soils causes chemical modification which depends on the soil type (effects for various soils types in table 7-18). Downside is that lime stabilized soils may be subject to durability problems caused by freezing/thawing therefore different conditions should be tested in lab before use: water can cause reduced strength; freeze/thaw cycles result in reduced strength; leaching reduces the Ca content and reduces effect on PI; carbonation due to atmospheric CO_2 causes $CaCO_2$ to form reverting the initial beneficial effects of lime stabilization; Sulphate present in groundwater/soil can combine with Ca to form compounds with over 200% increase in the volume causing massive differential swelling;

Cement stabilization is used for low-PI clays, sandy and granular soils to improve strength and stiffness. Typically cement content varies between 3-10% resulting in a unconfined compressive strength of 1MPa. Cement types I, II and III have all been used. The presence of organic compound and/or sulphates can have detrimental effects on cement stabilization. Organic matter such as undecomposed vegetation inhibits the normal hardening process resulting in reduced strength (can be tested with 10:1 weight mixture of soil and cement after 15 min, the pH should be at least 12 then organics are not problem). Sulphates are a problem only when clay is stabilized with cement.

Stabilization with Lime-Flyash (LF) and Lime-Cement-Flyash (LCF) or also known as stabilization with coal ash is used to stabilize coarse grained soils. In this case % of LF, moisture content, ratio of lime to flyash can be varied and is therefore harder to optimize for best results.

Asphalt stabilization soils are used for base/subbase construction. Asphalt stabilizing effect depends on the soil: sand-bitumen produces strength in cohesionless soils (e.g. sand); soil-bitumen stabilizes moisture content of fine-grained soils; sand-gravel-bitumen provides cohesive strength and waterproofs gravelly soils. Treatment of soils with over 20% fine content is not recommended. Treatment with asphalt coats soil particles which reduce water penetration (and strength decrease). In non-cohesive soils the second mechanism is increased adhesion as asphalt acts as binding agent. Three typical types of asphalt stabilization: sand-bitumen, gravel-bitumen and bitumen-lime. The type selection is based on construction and soil type and water conditions. Asphalt stabilization is very little affected by freeze/thaw cycles. Different gradation materials can be used with bitumen for optimal results (table 7-19 and 7-20).

Stabilization with Lime-Cement and Lime-Bitumen has the advantage that one component compensates for downsides of the other and vice-versa. The main purpose of lime is to increase workability and reduce PI of the soil and typically a few % is enough. Admixture design (for all above admixtures) follows the steps: classify soil; prepare trial mixes; develop density-moisture relationships; prepare triplicate samples and cure at target density; determine index strength; determine resilient modulus for optimum % admixture; freeze/thaw tests; select % for design requirement and add ca 1% to compensate for non-uniform mixing.

Soil encapsulation is foundation improvement to protect moisture sensitive fine-grained soils from moisture content variations by keeping the moisture below optimum. The encapsulated layer is surrounded by asphalt emulsion (and potentially geosynthetic layer) from all sides. Not recommended due to issues with maintenance.

Lightweight fill can be used in place of thick granular aggregates in soft subgrade conditions to reduce settling of subgrade. List of materials used as lightweight fill (table 7-21) include crushed slag, geofoam, shredded tires, expanded shale and others.

Deep foundation (and other foundation improvements) are used in case poor subgrade conditions are too large to stabilize or remove. Ground improvement aims at: increasing bearing-capacity and density; control deformations; accelerate consolidation; decrease imposed loads; provide lateral stability; form seepage cutoffs; transfer embankment loads to more competent layers. Methods described in table 7-22 and 7-23.

Recycle

Recycling of materials is an important concept, but the structural properties of recycled materials should be carefully assessed before usage. The two concepts are to reuse pavement materials or recycled waste materials. Both AC and PCC materials can be rubblized and compacted and reused as fill. Primary recycled waste materials e.g. tires, slag etc have the lightweight benefit.

Chapter 8 Construction and design verification of unbound pavement materials

After pavement design is completed the construction and quality control must meet the design requirements to achieve required pavement performance for the design life. Local conditions and materials can vary greatly in projects and although design has tried to take them into account there may be unforeseen conditions etc. during construction e.g. during subgrade stabilization.

Specifications

The specification dictates the quality for pavement section specified during design from contract. A set of standard specifications is recommended for typical regional conditions with corrective action measures in case of anomalies. Method specification detail the equipment and process to achieve a desired result. Result specification state what property must be achieved. Performance specifications details key material and construction quality characteristics which are correlated with long-term performance of the pavement. Main geotechnical intent of the specifications is to confirm the adequacy or improve the engineering properties of the soil or aggregate material (density, moisture control etc.). The problem with performance specification is that it does not detail the structural parameters of the lower pavement layers (which can also change in time) but affect the overall performance of the pavement system. All above specifications should emphasize raw material properties and should contain corrective measures in case of unsuitable soil conditions. These measures include: detection, depth of treatment, type of treatment and implementation responsibility.

Quality control (QC) and quality assurance (QA)

QC/QA practices are essential for obtaining satisfactory results of a pavement construction project. The QC/QA program is conducted either by the owner or a third party contractor who perform QC tests (Table 8-1) and provide QA documentation. A typical checklist for performing QC is in table 8-2 and depends on the specification type. *Proof rolling* can be implemented at any time during the preparation of the foundation/subgrade with the aim to point out soft and yielding material. *Test* such as nuclear density, sand cone, balloon or tube drive methods and moisture content determination should be performed regularly during the construction on each pavement layer. Emphasis should be on statistical characterization of materials. A valuable tool for QC can be process and measurement control performed *in-situ* over the period of different construction phases which could allow to determine how factors such has moisture content of subgrade affect compacted strength of a layer and point out the true cause of failings. *Chemical stabilization* requires additional controls such as pulverization and scarification, stabilizing agent content, uniformity of mixing, compaction and moisture control and curing. In general, measuring only density as measure of compaction is not sufficient and moisture content determination should always be included.

For embankment fills shear strength (slope stability) and consolidation (settlement potential) and for pavement structural layers stiffness and resilient modulus are used in engineering analysis as design intent. Construction practices however only determine density which is clearly insufficient to determine whether design intent has been met (and the correct parameter e.g. stiffness should be determined instead). Methods such as field CBR, DCP, FWD and/or laboratory tests should be included in QC. Proof rolling could be a potent method if coupled with lasers and digital video analysis to determine stiffness. Field CBR and plate-load tests could be used to verify if the design intent has been met for both flexible and rigid pavements (and are a standard technique in Europe). Downside is that they are time-consuming. Dynamic cone penetration (DCP) tests utilizes a cone to penetrate into the material with estimating the stiffness. It can be utilized fast and is mobile but can only tests points leaving gaps between points and requires good correlation methods with e.g. stiffness. Resilient modulus testing after earthworks could be performed with short Shelby tubes and lab testing with results available 1 day later. FWD tests is highly mobile and fast technique that measures deflection of the surface under falling weight and can be correlated with material modulus values to verify design intent. Other possible methods include geogauge and seismic methods which use seismic wave reflection on layers with different properties. Another emerging technology is the so called intelligent compaction which allows for continuous monitoring of pavement structure during rolling and provide to control in real time the design intent. Warranties typically for pavement construction have been 1 year but is shifting to be 2-5 years of constructor warranty.

Construction and construction monitoring

Construction of the pavement system involves grading to provide a uniform support layer(s) at required elevation. The aim is to achieve a structure with specific design intent. Good practice and common goal is to achieve the new alignment of the pavement with a balance of site materials (without transporting them from far away) i.e. it is best to have cuts and fills in balance. Typically, the first step is subgrade preparation by grubbing, removal of marginal soils, stabilization of in-place soils and improvement of drainage. Often removal and replacement is the easiest option, however saturated soils and high groundwater table should be considered before major earthworks. Topmost layers of soil containing organic matter should be removed throughout the alignment. Large embankments can cause settlement of existing soils thus examination of settlement potential is essential.

Drainage features should be employed as soon as conditions on site indicated drainage need (e.g. free water in subgrade, clogged underdrains in rehabilitation projects, saturated soils of high permeability, groundwater seepage, water seeping in test pits and high elevation cuts, water flowing on top of rock undercuts). Significant subgrade stability can be achieved with installation of underdrains (new projects) or clearing clogged underdrains (rehabilitation).

Excavation or removal of *in-situ* soil is required on most projects and accomplished with scrapers, excavators and haulers. Scrapers can remove material from one place and spread it in another (cuts and fills). Excavators need separate haulers to transport soil. During earthworks attention should be paid to rutting of the construction platform to identify potential subgrade problems.

Field compaction is defined as the densification of soils by application of mechanical energy often together with modification of water content. The goal is to achieve water content near compaction optimum moisture content (OMC) value for reduced volume change and increased strength. OWC should be determined in lab. Compaction equipment utilized depends on the soil type (table 8-4). Typically, compaction is done in series of lifts of depth 150-300 mm. For good compaction results a solid foundation is required (figure 8-11). The measure of compaction is density which can be measured on the site and related to density vs. water content curves. Optimal engineering properties of a given soil occur near OWC. Compaction is the cheapest and easiest to utilize method for soil improvement but its main drawback is that without proper QC deficiencies of compaction can go unnoticed. Compaction can be established using pressure, vibration or kneading action and there are several typed of equipment available for compaction: smooth drum roller (best for granular, non-cohesive soils), sheepsfoot roller (best for cohesive soils e.g. clays), pneumatic rollers and high-impact rollers which use ellipsoid shaped rollers for best effect. It is possible to measure soil response during compaction with sensors which acts as a continuous QC system.

Stabilization using chemicals, admixtures of flyash, cement or asphalt can be most economical to improve subgrade conditions. The basic steps include: pozzolan delivery and distribution, mixing, compacting and curing. Adequate mixing is essential for successful stabilization. Compaction is a standard procedure after stabilization, however care should be taken in case of lime stabilization which typically reduces maximum density and increases OMC. Also essential is adequate curing at temperatures above 4.4°C. All above stabilization methods require proper QA/QC.

Base and subbase construction require care when placing different aggregate layers to avoid segregation and contamination and thus loss of support/permeability. E.g. hauler wheels can carry saturated soils onto the aggregate layers or the aggregate material can be contaminated due to many transfer points. Spreading of the aggregate should be as uniform as possible. The aggregate materials are expected to be compacted, however failure to reach proper compaction can be due to different factors: lack of substrate support, improper compactor support, excessive moisture and segregation.

Pavement drainage system construction involves key points: stable foundation for construction of permeable base, quality of aggregate used as permeable base, prevention of fines entering the permeable base and stabilization of drainable base. The subgrade should be level, somewhat smooth and have a positive slope. Separation layers should be used in case of danger of fines entering permeable base from subgrade (can be an aggregate layer of geotextile). Proper edgedrains with required grades and materials are essential for a good drainage system. Care should be taken that edgedrains are not damaged during the construction. They should be backfilled with material with equal permeability as the permeable layer. At the end of construction the edgedrain system should be tested for permeability. Based on survey a good construction of the subsurface drainage system depends on a number of factors: expertise and knowledge of the constructor on drainage installation and presence of an expert during construction, a continuous path for water to drain, a positive slope, the pavement is supported by the system sets compaction requirements and care that other construction activities do not damage the system.

Performance monitoring

Pavement management systems can be used to track the pavement performance over time by performing distress surveys in periodic intervals. Utilizing only distress survey does not detect problems caused by moisture intrusion and are more effective when used together with other tests e.g. geophysical or FWD. Geophysical tests measure differences in material properties however these properties need interpretation and potentially wider knowledge of the region geology or baseline values for specific pavement sections. FWD can also be utilized for long-term monitoring of subgrade condition (pavement performance) by measuring at same locations and comparing the data with baseline values taken right after the pavement construction was finished. Drainage inspection is done with video cameras (video logging) very successfully to locate various problems with drainage systems. Geosynthetics allow to place sensors in them and monitor strains and deformations over long periods of time.

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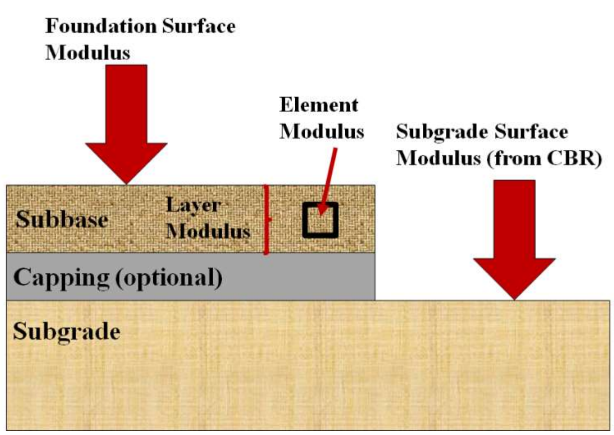
Design Guidance for Road Pavement Foundation (2009)

IAN73/06 should be read with HD 26/06. New design methods and criteria are based on classifying into four foundation classes. It includes to two forms: performance and restricted design. Performance design allows for a wide range of materials, methods and testing while restricted design is limited to schemes where performance testing is not appropriate.

The purpose of the foundation is to distribute vehicle loads onto the subgrade without causing distresses in foundation layers (during construction and service-life). The four classes are classified according to foundation surface modulus: class 1 (over 50 MPa), class 2 (over 100 MPa), class 3 (over 200 MPa) and class 4 (over 400 MPa). Covered materials include subgrade (natural or compacted fill), unbound or stabilized capping materials (series 600; specification MCHW1) and hydraulically bound subbase mixtures or granular subbase materials (series 800).

Restricted design (for classes 1-3) is conservative and intended for use in schemes of limited extent. Performance design covers classes 1-4 and *in-situ* the foundation surface modulus (measured immediately prior to placing overlying layers) is basis for the acceptance criterion for construction. The basis for choosing a class and design approach is economical with consideration for the project and available materials. Performance design allows for smaller layer thicknesses in case of higher quality materials. Frost action and drainage should be considered for both design approaches.



Stiffness modulus- ration of applied stress to strain.

Foundation surface modulus (FSM)- a measure of stiffness modulus at the top of the foundation (composite value of all underlying layers)

Subgrade surface modulus- an estimate value of subgrade stiffness modulus from CBR.

Layer modulus- an assigned stiffness modulus to a foundation layer.

Element modulus- an assigned stiffness modulus to a material sample (from lab tests).

Mean foundation surface modulus- the value that must be exceeded by moving mean of five consecutive FSM measurements (according to clauses 890, 891, 892 and 895).

Minimum foundation surface modulus- the value that must be exceeded by all individual measurements (according to clauses 890, 891, 892 and 895).

Groups of materials are established as: **unbound mixtures** (Series 800 e.g. clause 803; type 1 as clause 803; type 2 as clause 804; type 3 as clause 805 and clause 806; series 600 capping materials e.g. 6F-8f5 and 6s), **fast-setting** and **slow-setting** which achieve over and under 50% of specified strength in 28days at 20°C, respectively.

Foundation has a role both during construction and in service-life. During construction the foundation must withstand relatively high stresses of construction vehicles (although low traffic) and have sufficient durability against environmental effects. The foundation should also provide adequate stiffness for placement and compaction of overlying layers. Designs, drafts and materials given in draft HD 25. During the service-life the foundation must withstand large number of repeated traffic loads and environmental effects without deterioration of performance. The FSM class adopted for design must be maintained during the service life. Adequate maintenance (HA 44) and drainage should be provided during service-life.

The conservative **restricted foundation designs** should be used in cases when the range of tests required for performance design are inappropriate to be carried out. Bound subbase mixtures are restricted to CEM1 (EN197:1). During foundation construction the following information should be collected: strength measurements (CBR) at the top of subgrade and prior to placing an overlaying layer; material density and actual thickness of layers; compliance with relevant specification series 600 and 800.

The design requires the determination of the subgrade surface modulus from CBR, the lowest value of SSM from short and long term CBR should be used for design. The subgrade CBR should be determined on site before construction and must exceed the design CBR. In case the actual subgrade CBR is lower than the design value the subgrade must be improved (e.g. compaction) or foundation redesigned. The subgrade CBR should not be lower than 2.5% under no circumstances.

Required thicknesses for restricted design are given in figures 3.1 and 3.2. Foundation class 1 designs may use any capping materials from table 6/1 (series 600) and the finished surface must criteria for subbase in specification series 700. 4 options available for class 2 depend on whether unbound or bound subbase is chosen: granular subbase mixture to clause 803, 805 and 806; cement bound granular materials to clause 821 and 822; soil cement to clause 840. Cement based materials require compaction strength class of at least C3/4. Class 3 design are restricted to cement bound granular mixtures clause 821 and 822 achieving compression strength C8/10. Thicknesses should be rounded up to nearest 10 mm and tolerances according to series 700.

Development of **performance foundation designs** has several objectives: efficient use of various materials (natural, secondary and recycled) as binders and aggregates, to provide assurance that design requirements will be met; to account for structural contribution of foundation and allow for thinner pavement layers. Performance design relies on testing of physical properties critical to design. The FSM provides a convenient composite measure of the different layers of materials with varying properties. Pavement design in UK is based on principles of layered linear elastic modelling. The FSM provides a means to estimate the performance of the pavement (HD26) which can be used with all types of materials and layer thicknesses. Performance design predicts the FSM from different combinations of materials and layer thicknesses (figure 4.1) and should be used with clauses 890-896.

Subgrade requirements are identical to those described in restricted designs.

Foundation surface modulus requirements for mean and minimum values are in table 4.1 for different materials and design classes. FSM is measured with Dynamic Plate Test (DPT) in accordance with clauses 890-892 and 895 and involve both Falling Weight Deflectometers (FWD) and Lightweight Deflectometer (LWD). In case of LWD a correlation exercise of 25 measurements in the demonstration area should be done to compare it with FWD. The mean FSM provides assurance of the foundation performance in the short-term. The mean SFM requirement for slow-setting mixtures is lower than for fast-setting mixtures and it is also lower than long term modulus for unbound mixtures. Mean and minimum FSM values of classes 3 and 4 are likely not achieved economically with unbound materials.

Demonstration areas are required to enable the testing of the adequacy of the performance design of foundations. Demonstration area should be located where design DBR equal *in-situ* subgrade CBR (or subgrade CBR is higher). The mean and minimum FSM design values should be adjusted if in-situ CBR is higher because of unrealistic elevated FSM values. It is recommended that FSM testing is done at least after 24h or 7 days in case of unbound and fast-setting mixtures, respectively. It may also be advisable to measure surface modulus after placement of each layer. A trafficking trial (clause 891) is required with a limit to rut depth and that FSM values continue to meet mean and minimum FSM design values.

Performance testing (FSM) on main works should be carried out before laying of topmost pavement layers. During works material testing, density and thickness measurements in accordance with clauses series 600 and 800 should be conducted.

Three criteria are considered for layer thickness design in performance testing: protection of subgrade during construction, provision of adequate support stiffness for overlying layers and practical minimum layer thickness for construction. A large number of combinations of materials, layers and thicknesses that meet the design requirements are possible (figure 4.2-4.6). Subgrades with CBR below 2.5% cause problems and are not included (should be strengthened to that value). The minimum layer thicknesses are 150 mm for class 1 and 2, 175 mm for class 3 and 200 mm for class 4 foundations. Maximum layer stiffness (to exclude very stiff and very thin layers which are susceptible to cracking) are 100 MPa for class 1, 350 MPa for class 2, 1000 MPa for class 3 and 3500 MPa for class 4. Capping layer is only considered in design as a structural layer for class 2 since its provides little support (PPR127). Negative tolerances of 15 mm are suggested to avoid too low layer thicknesses (and potential failure). Values should be rounded up to the nearest 10 mm. The complete surface must meet the series 700 requirements.

Material characterization in the UK in the design phase is primarily done with the stiffness modulus. It can be problematic to measure consistently for subgrade therefore CBR is used as an indirect measure. Estimation of short and long-term CBR should be done in lab with different moisture and "disturbance" conditions. If lab tests are not possible Suction Index Method (appendix C of LR 1132) should be used instead. In case or large projects with varying soil conditions extensive survey is necessary. The selection of design CBR should be done with consideration to moisture content during construction and also long-term moisture conditions. Moisture Condition Value (MCV) is typically used to measure compaction and is measured during construction. It can be related to the ability of the subgrade to withstand traffic (soil strength). Conversion of CBR to subgrade surface modulus can be done as with equation on page 26.

In the construction phase the *in-situ* CBR should be checked against the design CBR in case of both restricted and performance designs. Dynamic cone penetration (DCP) to determine CBR should be carried according to clause 893 of draft specifications and should include at least 5 tests in demonstration area and tests along the main works should be no more than 60 m apart.

The minimum allowed design CBR is 2.5%. Subgrades with lower CBR are not suitable for foundation support and should be improved by replacement, treatment, compaction or drainage. Removable layer is typically between 0.5 and 1 m and should be assigned CBR of 2.5% (even if measured higher) to account for possible soft material underneath. In case of cohesive soils lime stabilization may be appropriate (HA44 DMRB 4.1.1. and HA74 DMRB 4.1.6). For certain condition the incorporation of a geosynthetic material into the foundation can be beneficial.

Performance tests require estimation of long-term layer modulus values for materials in each layer. Table 5.2 lists methods to assess elemental modulus. Dynamic plate testing (DPT) has the advantage that it can be used to assess performance design foundations and for material testing is conducted in either a small trial site or in the lab. Standard test for HBM mixtures is modulus of elasticity testing (BS EN 13286-43). However, lab tests overestimate the real value and for fast-setting mixture 20% and for slow-setting mixtures 10% of lab test value should be used for design. Also, layer stiffness modulus of HBM may not meet the surface modulus requirements due to soil and material variation. Springbox testing allows to test small samples under realistic stresses and appropriate moisture conditions. Stiffness of an unbound material depends on the moisture content (lower stiffness with increased moisture) and can be strongly affected by weather conditions during construction. Unbound material stiffness is also increased when confined by an overlying layer and results in higher stiffness values during service-life that during construction. Some hydraulically bound mixtures (HBM) are slow setting which results in significantly reduced stiffness during construction. DPT during reconstruction can be used to derive FSM values via back-analysis and it is not necessary to distinguish between stiffness moduli of separate layers (HD29 DMRB 7.3.2).

Employing adequate **drainage** to keep water out of capping, subgrade and subbase during construction and service-life is important. During construction the subgrade should be protected by constructing foundation layers before e.g. rain can soften it or remove excessive water by installing drainage. Infiltration of water into the pavement system should be minimized in the long term and adequate drainage constructed. If possible foundation drainage should be kept separate from pavement run-off drainage and adequate sloped should be employed. In reconstruction projects continuation of drainage should be maintained. A granular drainage blanket (MCHW 1 series 600) of thickness between 150 and 200 mm could be used to drain water infiltration from surface. In case of high water table and/or moisture sensitive materials slot drains placed below capping or subbase can solve problems efficiently. Permeability of subbase and capping materials should be determined and checked if it is sufficient to remove infiltrated water unless underlying materials (subgrade) is more permeable and the water table is more than 300 mm below the foundation.

All materials within 450 mm of road surface should be non-frost-susceptible (series 600 and BS812 part 124).

Pavement testing is done to check compliance with specification or for pavement performance assessment (HD30 DMRB 7.3.3). Density testing can be carried out in-situ with a nuclear density gauge where a radiating source is applied to the material and the radiation is measured a distance away and decreases with increasing density. Moisture content is measured with same apparatus but different radiation which is absorbed by hydrogen atoms. The testing is fast (ca 5 min) with portable equipment but calibration for each soil is required. Density can also be determined by time-consuming volumetric tests.

The CBR test involves insertion of a plunger into the ground at a rate of 1mm per minute and recording the load. The load at penetrations of 2.5 and 5 mm are compared with results of standard aggregates and ratio given as %. The CBR is a combination of stiffness modulus and shear strength, is not suitable for coarse aggregates and takes 0.5h on site and 1-2h in lab.

Static cone penetrometers can be used to rapidly assess the approximate CBR of soft and medium fine grained subgrades. DCP uses a falling weight onto an anvil and can be used on coarser materials than other penetrometers. The rate of penetration can be related to CBR (HD29 DMRB 7.3.2 and clause 893). DCP uses a 8kg drop weight falling vertically through 575 mm and hitting a relatively light anvil. Subgrade assessment is done by recording distance in mm per 1 blow between 50 and 550 mm penetration from top of subgrade. DCP can be used to measure CBR and thickness of composite foundations.

Plate bearing test is used to determine CBR of coarse materials (BS1377 and MCWH 1 series 600). Tests are time consuming and equipment is heavy. Dynamic plate tests involve dropping a weight onto a plate and measuring deflection which can be used to derive stiffness modulus. Lightweight falling deflectometer is used in case of stiffer materials.

Springbox equipment is a suitable tool for testing unbound granular and some weak HBMs. Repeated vertical loads are applied to material in a steel box and deflections are measured both horizontally and vertically and related to the stiffness modulus.

Annex A: Equations of thickness of performance design.

Annex B: Procedure for alternative performance foundation design

Alternative design options may be calculated analytically with multi-layer linear elastic analysis and show that the design criteria are met. Inputs are given in figure B1. Limits on the maximum allowed strain of subgrade vary with subgrade stiffness modulus (figure B2). Adequate support for pavement during service-life is defined by maximum allowed deflections of foundation for different design classes.

Annex C: Performance design flowcharts and examples

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Geotechnical Technical Guidance Manual (2007). U-S Department of Transportation. Federal Highway Administration.

Section1 Introduction

This geotechnical technical guidance manual (TGM) provides guidance for applying policies, standards and criteria to manage finances and public safety and in case standards do not exist. It should be used in conjunction with project development and design manual (PDDM; chapter 6). Exhibit 1 shows relationship of TGM with other guidance, references and procedures.

Geotechnical discipline

The TGM focuses on geotechnical works done by Federal Lands Highway which is unique because most of the work deals with low volume roads on resource sensitive public lands. Firstly, budgets of such projects are smaller and cost-efficient project design are required but more importantly care must be paid to management of safety, cultural, preservation environmental and natural resources (e.g. wildlife, scenic beauty). The geotechnical works and goals should: respect the land, local values, wildlife and habitat; provide a safe passage for residents, visitors, wildlife etc.; blend the improvements into the setting; accomplish within budget constraints. Geotechnical works need to be cost-efficient and differ from those done of interstate highways.

Geotechnical role in project development

Typically, each project is carried out by a cross-functional team and the geotechnical experts are involved in following: structures (primarily walls and bridges); significant earthworks (cuts and fills); geohazards (landslides and rockfall); seismic hazards; problematic soils (frost and swell susceptible soils). A typical chronology for geotechnical involvement: project initiation and scoping; survey available geotechnical data; perform field recon, preliminary and supplement investigations; summarize data; perform main geotechnical investigations and prepare reports; provide plans, specifications and estimates (PS&E) support; provide support during construction.

During **preliminary work** of project definition geotechnical recommendation is at the conceptual level and involves searching for existing data and performing site recon and can include a few key borings for critical and/or complex projects. The aim at this stage is to identify and avoid critical flaws, hazards and identify possible geotechnical issues. Done typically before 30% plan level.

Geotechnical investigation includes earthworks, bridges, walls, landslides and material source investigation. Activity tasks include: site investigation plans; drill permits; coordination with partners; schedule and perform site investigation; field boring logs; lab testing if necessary; evaluate constructability issues. Conducted typically before 70% of plan-in-hand delivery.

Geotechnical recommendations involve activities such as: summary of soil/rock testing and boring logs; plan and profile of subsurface investigations; geotechnical analysis and design; conclusions and recommendations together with the final report. This part also includes technical issues related to cuts and fills, slopes, walls and bridge designs (e,g, types, size and location TS&L). Conducted typically before 70% of plan-in-hand delivery.

Geotechnical PS&E consists of conduction geotechnical reviews of PS&E and is conducted typically between 30 to 70% of plan-in-hand delivery. The final design of geotechnical project features is established in this phase.

Construction support is about evaluating geotechnical evaluation of risks and issues that arise during bid process, contractor submittals and during construction.

Use of the TGM

The TGM is intended to be used by geotechnical experts at the FLH i.e. for typically construction of low-volume roads. The TGM should be used together with PDDM on applying policy and standards. The TGM most important aspect is to provide highest level "how to" guidance. Appendix A lists all cited sources.

Section 2 Guidance and references

Policies for the geotechnical discipline

Policies are guiding principles that should be followed without exception thus they are quite general. The typically problems and issues as well as delivery objectives were introduced in the 1. Section (under geotechnical discipline). The combination of limited funds, protecting local resources and local stakeholders (with local policies) means that geotechnical solutions must be context-sensitive and cost-effective thus require identifying and planning appropriate level of geotechnical practice for each unique project. The highest level of geotechnical guidance is provided by seven geotechnical policies: 1) support the mission, vision and program management objectives of the agency 2) meet the technical scope requirements as provided by PDDM 3) advance the state of practice by implementing new technology 4) demonstrate environmental stewardship in investigations and design 5) demonstrate financial, cultural and natural resource stewardship 6) conduct work safety and seek safety improvement solutions 7) achieve quality through established QA/QC. The basis documents for each of these seven policies are given on pages 2-2 to 2-6. Federal laws and policies that influence the above policies are in exhibit 2.1 and on pages 2-6 to 2-10.

Risk management

Risk is inherent in geotechnical projects e.g. with respect to cost when deciding the extent of geotechnical investigations. Larger investigations generally reduce risks encountered during construction and service-life. Risk assessment is an important part of each project and careful consideration should be given to the risk severity vs. the benefit of reducing the risk(s) involved. In case when standards and standard projects are used risks assessment does not require attention.

Standards and standard practice

The general standards are provided by the AASHTO guides but in low-volume roads implicit geotechnical exploration according to AASHTO may be impractical and not economical. In those cases simple practical risk assessment and use of e.g. local standard solutions can help reduce project costs. The highest cost in exploration programs is associated with drilling thus it might be economical to use non-invasive methods for most of the exploration program and supplement that information with borings at key locations. Risk assessment is advised.

Geotechnical standards and standard practices involve sampling, testing, analysis, design and reporting. Standards are based primarily on what has worked in the based with respect to risk management, quality and efficiency. Standards describe an efficient (time and cost) solution which works at certain conditions. On the other side, over-standardization can lead to inefficient designs, lack of innovation and insensitivity to project conditions so engineering expertise is required when opting for a standard or non-standard solution.

Technical guidance

TGM provides guidance for where standards exist and in case when a standard is not applicable. In such cases, TGM provides previous experience on the “how to” in the form of practices that have been successful.

Technical and state DOT references

References in the TGM are classified as primary and secondary. Primary sources provide preferred guidance on hoe to accomplish a task or are the most widely available source. Secondary sources are additional documents that supplement the primary sources.

State Departments of Transport have local policies and manuals based on local climate etc., these are listed in exhibit 2.6.

Section 3 Geotecnical investigations

The primary aim of geotechnical investigations is to provide the design and construction engineer with subsurface and material conditions at the site. Because the conditions and projects can vary drastically the geotechnical investigations do not follow a rigid format but the field data and analysis is critically important for all subsequent decisions. Following fundamental information should be obtained: identification and delineation of soil and rock strata; condition and performance of existing transport structures; engineering properties of soil and rock strata; groundwater levels; slope stability and other geologic hazards/constraints; environmental concerns. The most expensive component of geotechnical investigations is drilling and if possible should be minimized by use of test pitting and geologic interpretations.

Planning and management

Reference for planning the scope of geotechnical exploration is the NHI 132031 (AASHTO MS-1) but due to varying conditions engineering judgement should be used. The first step is a through review of project requirements: project size and type; project criteria and constraints; project design and construction schedule (exhibit 3.1-A). The scope and cost of the geotechnical investigation should be made using engineering judgement to fit the variability, conditions, cost and constraints of the project so that a suitable design can be proposed (with valid assumptions and engineering value for materials). Also, the potential for failure and consequences should be considered. A comprehensive program should start with review of available data on the project location and geology etc. and followed by site recon to identify landforms and geologic conditions so that the subsurface exploration plan can be optimized.

The amount and type of geotechnical data collected is a balance between available finances, manpower and equipment and the complexity of the construction project i.e. what would be the costs of more conservative design vs. exploration costs and what are the potential risks of less geotechnical testing. The investigation plan should be sufficient to minimize “changed conditions” claims and modifications during construction. Subsurface exploration should use the phased approach and when soil conditions vary/differ from those assumed during planning, extra tests should be scheduled. This can include boring depths, number and type of samples, number and location of explorations. Planning the number and extent od explorations should be according to PDDM and FHWA-ED-88-053 and table/exhibit 3.1-B for minimum boring criteria. For low-volume roads exhibit 3.1-C is a reasonable guideline for cost-effective testing. For example, boring can be supplemented with geologic mapping and surveying, geophysical techniques and test pits. Prior knowledge of the geology can also be invaluable. Finally, the type and number of lab tests should be established according to factors: project scope; problem soils and variation; proposed foundation types and loads; seismicity; settlement constraints; height and slope of cuts and fills.

A vast majority of projects are **roadway alignment investigations** where survey explorations are made along the proposed roadway alignment to gather information about: design cut and fill slopes; assess material suitability for embankment; define limits of borrow materials; asses foundation material suitability; evaluate settlement and slope stability; quantify topsoil to be removed; design remedial measures in case of poor soils; aid design the pavement system. Typical exploration extends along the centerline (but can be zig-zagged around the centerline). Sampling should allow for classification, gradation, moisture content and Attenberg limit tests. Undisturbed samples should be collected in case of unusual strength and consolidation. Additional borings should be made in the transverse section in case of variable conditions. Rock cores should be obtained in case of rock subsurface.

Material source investigations are conducted with the aim to evaluate the quantity and quality of available and potential material sources in the vicinity of the project. These material can include gravel base, crushed surface materials, pavement and concrete aggregate, backfill and borrow materials. Economic risk involved with developing new sources should be assessed. Minimum determined information should include: expected quality and procedure to obtain that quality; boundary limits of materials; specific areas and elevation of unsuitable materials; previous uses of material from the source; use and limitations of materials; potential development, processing and handling problems; legal description of the site location; regulatory permits and reclamation requirements. Expansion of existing sources and development of new material sources requires additional information: determine preliminary rock quality; review site geology, aerial and contour maps; review FWHA-ED-88-053 checklist for material investigation; a recon level of review of material source site; exploration permits; exploration equipment; groundwater level recording; samples for classification, moisture, compaction, permeability and corrosion; lab testing of representative samples.

Structure foundation investigation can include bridge structure, retaining structure, building, tunnel and mast support investigations. All structure investigations should include SPT at regular intervals. Undisturbed samples of cohesive soils should be tested for shear strength and material properties (moisture, gradation, unit weight, Atterberg limits). Groundwater level must be determined. In case of building one boring should be made at each corner and one in the middle, for large building each support location should be investigated.

Landslides and mitigation measures should be monitored over a minimum period of several months with inclinometers and piezometers to determine the depth and size of movement in the zone. Minimum of two boring must be drilled along the axis of movement of the slide and should extend the full depth of the landslide material (minimum of 5m into stable material). Groundwater heads can be located with piezometer equipment and the landslide survey should cover more that the expected extent of the landslides to provide sufficient information of the countermeasures.

Safety concerns regarding the climatic conditions and utilized equipment should be evaluated and safety plans developed (PDDM section 6.3.1.3). Minimum safety gear should include hard hat, gloves, goggles and safety boots. Before the subsurface exploration it may be necessary to check for the presence of utilities under-ground. Environmental/climatic factors can also affect the schedule and cost of a project.

Methods and practice

Preliminary study and recon should be carried out for project definition phase at a conceptual/feasibility level and should extend to probe materials and conditions and identify potential hazards and risks. Investigations typically include site recon by a geology expert and gathering/review of existing data, maps, records etc. at the office. A few boring could be necessary at critical locations. At this stage it is possible to evaluate alternate locations for the roadway alignment which could result in significant cost savings. Potential sources of available data on the location are shown on exhibit 3.2-A. The aim of **field recon** is to evaluate geographic, topographic, geologic and geotechnical issues and hazards along the roadway alignment route(s). Key site locations and conditions should be photographed and documented as well as landslides, faults, springs, rockfall and erosion identified. For best design concepts the recon should be performed with full understanding of project requirements. Field recon should define the following factors: stratigraphy; key outcrops; existing slope assessment; ground and surface water; geologic constraints (conditions that adversely affect the project outcome); environmental considerations; exploration type and extent; drilling logistics; requirement for permits. Guidelines for documentation of field recon are shown on exhibits 3.2-B and 3.2-C.

Surface exploration involves three main methods: field recon, geologic mapping and field developed cross-sections. Field mapping should start by observing road cuts, drainage courses and bank exposures and a large-scale topographic map is essential. The main aim is to confirm main types of rock and soil that are present and note any features that assist engineering analysis (e.g. angle and performance of slopes). In case of rock slopes these should be identified and exhibit 3.2-D shows an example of rock mapping documentation. Field-developed cross-sections are applicable to any geotechnical investigation: placement of materials, foundations and slopes, borrow and aggregate materials, analysis of slope stability, seismic and drainage activity. Field mapping should also include interpretations of surface and subsurface materials relationships as well as estimates for engineering parameters which would help develop the full subsurface exploration program.

Subsurface exploration methods can be used after obtaining right-of-entry and utility clearances. Guidelines for use of heavy, specialized geotechnical equipment are shown on exhibit 3.2-E. Geophysical methods (exhibit 3.2-F) are used to gather geologic information from a large area to help to supplement bore hole data. The limitations of geophysical methods are that no samples are recovered, interpretation is complicated and depends on the presence of groundwater and inhomogeneity of subsurface layers. Therefore, geophysical methods should be secondary characterization methods.

Drilling and sampling activities should be done according to AASHTO and ASTM standards and detailed list of drilling methods is in NHI 132031. The aim is to log accurately the conditions and obtain samples of subsurface for lab testing. Disturbed samples can be used for determination of general classification, lithology of soil deposits, identification of soil components, grain size, Atterberg limits and compaction characteristics. The most common method is the SPT with common errors on exhibit 3.2-G.

Undisturbed samples are obtained with special methods to prevent segregation in fine-grained soil strata for lab testing to determine engineering characteristics.

Rock samples can be obtained for outcrops and test pits while rock cores are obtained from drilling operations using single, double or triple tube barrels according to AASHTO and ASTM. In some cases the rock orientation is important and can be determined with special barrels or by video/photo surveying rock boreholes which can also be used for identification of fracture and shear zones.

Test pits, trenches and surface exposures are the simplest methods for observing of subsurface soils performed by hand, backhoe or dozer excavations. They provide detailed soil data at low cost and sampling speed at shallow depth. In case of significant soil variation exploration pits can be critical. Drawback is the depth limitation and that they can potentially cause a large area of disturbance. Guidance in NHI 132031 (AASHTO MS-1). They should be backfilled with excavated material and in case of e.g. agricultural soil backfilling should be in correct order. In fact all borings and test pits should be properly closed after end of exploration (NHI 132031). In case groundwater is encountered the closing should be a mixture of powdered bentonite, Portland cement and water.

Care and retention of samples should comply with NHI 132031 (AASHTO MS-1). The obtained samples retain a significant amount of effort, time and money and should be properly labeled. The labelling should include: project name and number; box number of total set; bore hole and sample number; depth data; date; name of personnel. Typically, the samples are either disturbed, undisturbed or rock samples and should be held until construction is finished.

Exploration issues and difficulties may be encountered during exploration due to site-specific geology or wrong equipment/method. All difficulties during drilling and sampling should be included in the final logs.

Soil and rock classification

Consistent approach to identifying subsurface materials is described in standard PDDM and guidance in NHI 132031. Material descriptions are based on visual inspection while material classifications are based on lab index-tests. Classification should be as complete as possible on factual information available. Soil classification order of soil samples (exhibit 3.2-H): apparent consistency and density, color, secondary, primary and additional soil constituents, moisture content and geologic name or formation. Rock classification order: type, color, grain size and shape, stratification, mineral composition, weathering and alteration, strength, hardness, discontinuities, rock quality designation and formation name.

Exploration logs

Standard is to produce a log for every performed exploration (boring hole, test pit etc.) referred in NHI 132031. Groundwater levels should be measured each morning at test locations. Each log should provide basic data on the drilling but also describe each sample fully by recording the depth of each stratum, discontinuity, lens and the reason for terminating each hole. **Field log** should contain all the information from each exploratory hole with descriptions of each obtained soil/rock sample. Top and bottom of each stratum should be included together with groundwater level. **The final log** should be complemented with lab test results and factualized descriptions of materials, conditions and strata encountered during drilling according to NHI 132031 (AASHTO MS-1).

***In-situ* soil testing**

In-situ soil testing is beneficial when obtaining representative samples for labs tests is not feasible or is difficult (in case of soft soils or loose silts and sands) and it produces very little on-site disturbance. *In-situ* tests include correlation tests, strength and deformation tests and permeability tests. Data from correlation tests can be correlated with design parameters such as density and shear strength according NHI 132031 (AASHTO MS-1). Most typical test is the SPT which is easy and inexpensive to use and provides a sample at each location. SPT test values should be used with care for correlations and possibly corrected for different materials as shown on exhibit 3.2-I. DCP should be used as supplementary method to SPT.

Various strength and deformation tests exist. Cone penetration test (CPT) and piezocone penetration test (PCPT) are specialized quasi static profiling tests for sands and clays (nor rock or hard soil). Since no samples are collected should be used to supplement other methods. Pressure-meter test (PMT) measures stress/strain by inflating a probe at a certain depth in a borehole. Horizontal stresses, shear strength bearing capabilities and settlement can be estimated from empirical correlations. Dilatometer test (DMT) uses pressure readings from a flat plate at the base of a borehole to provide stratigraphy, lateral stresses, elastic modulus and shear strength of loose to medium sands. Requires calibration. Field vane hear test (VST) consists of pushing a four-bladed vane at the borehole base into very soft cohesive soil of organic deposit and applying a torque to provide undrained shear strength. Borehole shear test (BST) is performed in an uncased borehole to determine shear strength by measuring the resistance of pulling up the horizontally expanded device along the borehole.

Most commonly used permeability tests include the pumping and the slug test. Pumping test requires a pumping well and 1- 4 adjacent well for observing the changes in water level. Slug tests creates a rapid change of the water level in a well wither by removing or injecting a large amount of water and observing the water level. Additional tests include packer, infiltration and open borehole seepage tests.

Laboratory testing

Sufficient lab testing should be performed to represent the actual site conditions. The type and number of tests required is primarily determined by site condition variability, investigation purpose and the amount of risk and potential failure consequence. Exhibit 3.2-J can be used as guideline. Engineering judgement should be used to plan a cost-efficient program that would provide sufficient data for design purposes. Soil and rock tests should determine engineering properties of materials and their response to varying conditions (NHI 132031 and exhibits 3.2-K and 3.2-L).

Instrumentation and monitoring

Geotechnical are used to characterize site conditions, verify design assumptions, monitor construction effects, enforce quality and provide early warning of failure. They should be used to answer specific questions and provide engineering insight. A successful instrumentation program should include a plan according to: objectives; identification of instruments; site plan; acquisition, calibration and installation of instruments; personnel training; monitoring and data analysis; documentation and follow-up. Instrumentation set up on the site is vulnerable to weather but also to vandalism, care should be takes to reduce such risks of damage. Proper maintenance is required. Data analysis should follow standard practice as designated by AASHTO and FHWA. Graphs of the variable of interest against time provide an overview of the collected data.

LISA 7

ERINEVATE RIIKIDE NORMIDE LÜHIÜLEVAADE

Muldkeha projekteerimine (sh arvutused) ja tüüplahenduste väljatöötamine geosünteetika kvaliteedikontrolli arendamine

ERINEVATE RIIKIDE NORMIDE MULDKEHA OSA LÜHIÜLEVAADE

JUULI 2020

Arendustöö

KOOSTASID: **PILLE SEDMAN**

PEETER TALVISTE

TALLINN 2020

SISSEJUHATUS

Käesolevas ülevaatega antakse võrdlev analüüs mulde ja katendi projekteerimise põhimõtetest erinevates riikides. Eesmärgiks on koondada teadmiste baas otsustamaks teede projekteerimise tuleviku üle Eestis.

Ülevaade on koostatud järgmiste (aga mitte ainult) juhendite alusel:

1. Suurbritannia.
    a. Highways England, Design Manual for Roads and Bridges. Series 600 Earthworks, Series 700 Road pavement – general, Series 800 Road pavements unbound, cement and hydraulically bound mixtures.
    b. CD 225 Design for new pavement foundations (mulle v muldkeha)
    c. CD 226 Design for new pavement construction
2. USA
    a. NHI Course No.132040. Geotechnical Aspects of Pavements. Publication No. FHWA NHI 05-037.
    b. AASTHO Pavement Design Guides (1961, 1972, 1986, 1993, 1998)
    c. NCHRP 1-37A Pavement Design Guide
3. Venemaa
    a. BCH 46-83 ja ODN 218.046-01
4. Eesti (Vene ja Eesti süsteemi võrdlus teostati 2017 aastal)

SUURBRITANNIA (UK)

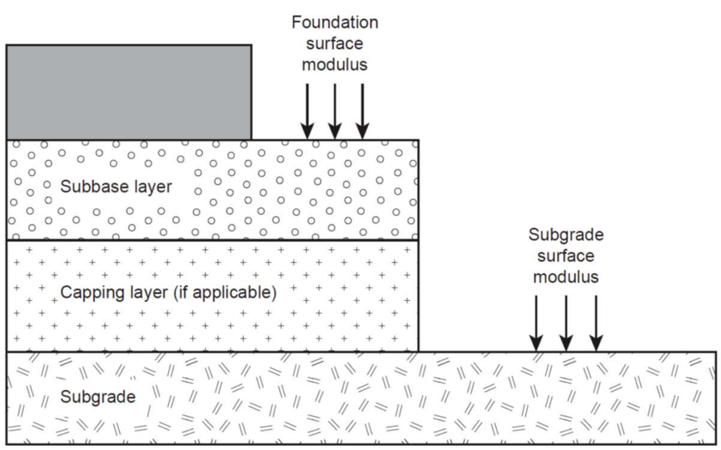
Muldkeha projekteerimine
CD 225 Design for new pavement foundations (Aprill, 2020)

Projekteerimise eelduseks on, et konstruktsioon on hästi dreenitud. Muldkeha rajatakse aluspinnasele (sub-grade) ning see koosneb vajadusel tugevduskihist (või vahekihist - *capping layer*) ning katendi aluskihist (*sub-base*).

Projekteerimisel on kolm lähenemist:

- tüüplahendus – ehk projekteerimine kasutades piiratud valikut hästi tuntud spetsifitseeritud materjalidest;
- tulemusprojekteerimine – mis võimaldab kasutada erinevaid materjale tingimusel, et materjali kasutamisel saavutatav vundamendi pinnamoodul (*foundation surface modulus* – pinnamoodul kattekihtide all) määratakse enne kattekihtide paigaldamist koha peal mõõtmisega katseväljakul ja põhitööde käigus kontrollimaks vastavust nõuetele (projektile);
- tee laienduse meetod – tee laiendamise korral kasutatav lähenemine, kasutatakse kas tüüplahenduste meetodit või tulemusprojekteerimist, arvestades samal ajal lisanõudeid olemasoleva kuivenduse järjepidevuse tagamiseks.

Muldkeha projekteerimine põhineb aluspinnase pinnamoodulil (*subgrade surface modulus*), mis määratakse kohapeal, laboris või hinnatakse. Peamiselt kasutatakse CBR-testi (*California Bearing Ratio*), mille tulemus konverteeritakse nn. taastuvaks elastsusmooduliks (M_R – *resilient modulus*, sisuliselt E_{V2}). Eristatakse aluspinnase lühiajalist pinnamoodulit (ehitusaegne) ja pikaajalist pinnamoodulit (ekspluatatsiooniaegne, kujuneb valminud konstruktsioonis koormuse all). Nendest väiksem valitakse projektpinnamooduliks. Seega – aluspinnase projektpinnamoodulina käsitletakse aluspinnase vähimat taastuvat elastsusmoodulit (M_R), mis reeglina määratakse uuringutega. CBR-test ei ole ainus M_R määramise meetod.



Kui aluspinnase (projekt)pinnamoodul jääb alla 30 MPa, siis tuleb aluspinnast parendada. Kui aluspinnase (projekt)pinnamoodul on alla 50 MPa, siis tuleb kasutada vahekihti.

Muldkeha projekteeritakse vastavalt pikaajalisele vundamendi pinnamooduli klassile, milleks on:

1 klass – vundamendi pinnamoodul E ≥ 50 MPa, tavaliselt on kasutusel vaid tugevduskiht (*capping layer*),

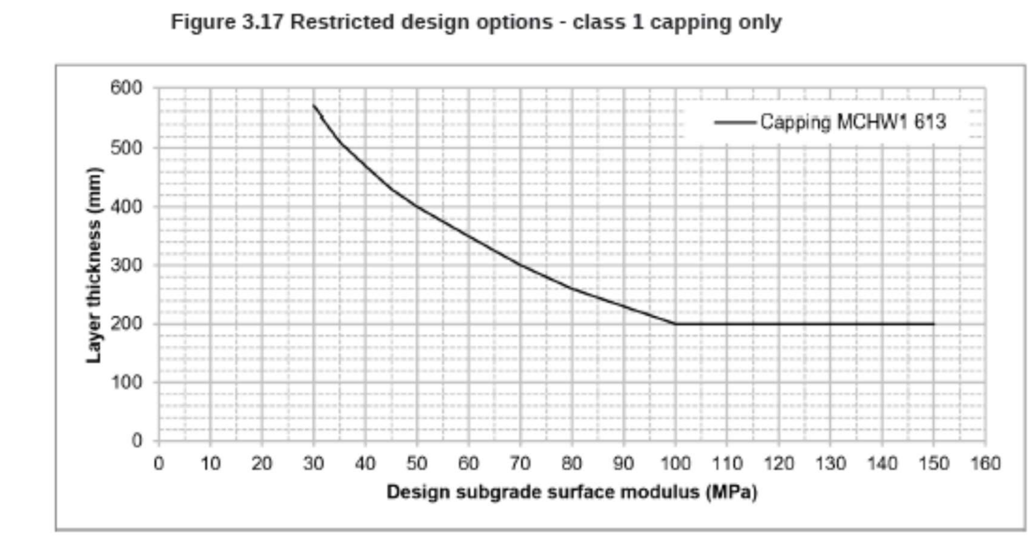
2 klass – vundamendi pinnamoodul E ≥ 100 MPa, tavaliselt on kasutusel aluskiht (*sub-base*), vajadusel ka tugevduskiht.

3 klass – vundamendi pinnamoodul E ≥ 200 MPa, tavaliselt on aluskiht nõrgalt stabiliseerituna (*including hydraulically bound*)

4 klass – vundamendi pinnamoodul E ≥ 400 MPa, tavaliselt on aluskiht tugevalt stabiliseerituna (*including hydraulically bound*)

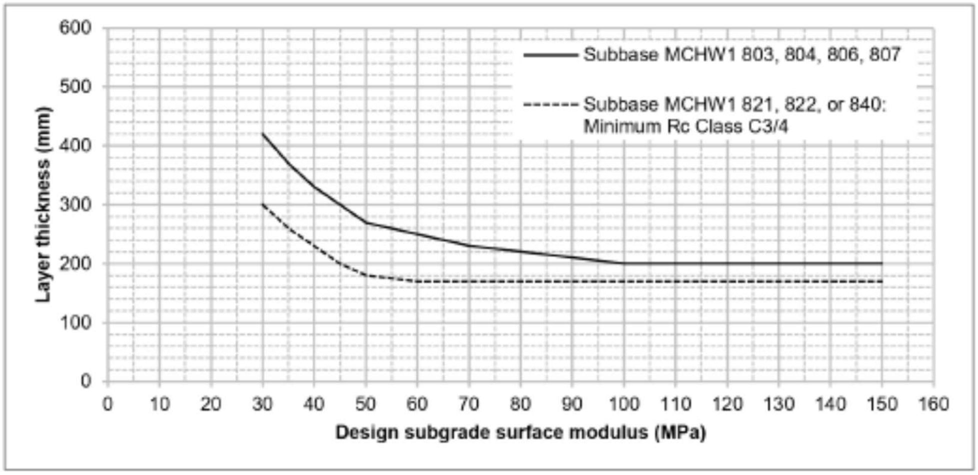
Tüüplahenduste järgi projekteerimine

Tüüplahendused on välja töötatud katselõikude andmetele tuginedes. 144 reaalselt katselõigult on kogutud andmed ning tüüplahendused kehtivad konkreetsete materjalide kohta. Materjalide nõuded on esitatud spetsifikatsioonidena. Sõltuvalt aluspinnase pinnamoodulist saab nonogrammidelt määrata vahekihi (*capping layer*) ja katendi aluskihi (*sub-base layer*) paksuse vastavalt muldkeha klassile (Joonis 1, 2).



Joonis 1. Väljavõte CD 225 revision 1 Design for new pavement foundations (Aprill, 2020)

Figure 3.18 Restricted design options - class 2 subbase only



Joonis 2. Väljavõte CD 225 revision 1 Design for new pavement foundations (Aprill, 2020)

Tulemusprojekteerimine

Kihtide paksused saadakse analüütiliselt mitmekihilise lineaarse elastse poolruumi mudeliga kasutades kihi moodulit ja miinimumpaksust (sisuliselt on kasutusel Odemark valem). Kasutada võib ka valmis nomogramme (sarnaselt BCH 46-83 ja ODN 218.046-01).

Kontrollida tuleb aluspinnase deformatsiooni (ehitusaegse koormusega) ja mulde deformatsiooni (pikaajaline) ratta koormuse all. Peavad vastama nõuetele.

Muldkeha pinnamooduli vastavus tuleb tõestada katseväljakul ja edasi tööde käigus vastavalt ettenähtud protseduuridele.

Katendi projekteerimine

CD 226 Design for new pavement construction

Tüüplahendustega projekteerimine

Lubatud materjalid on spetsifitseeritud.

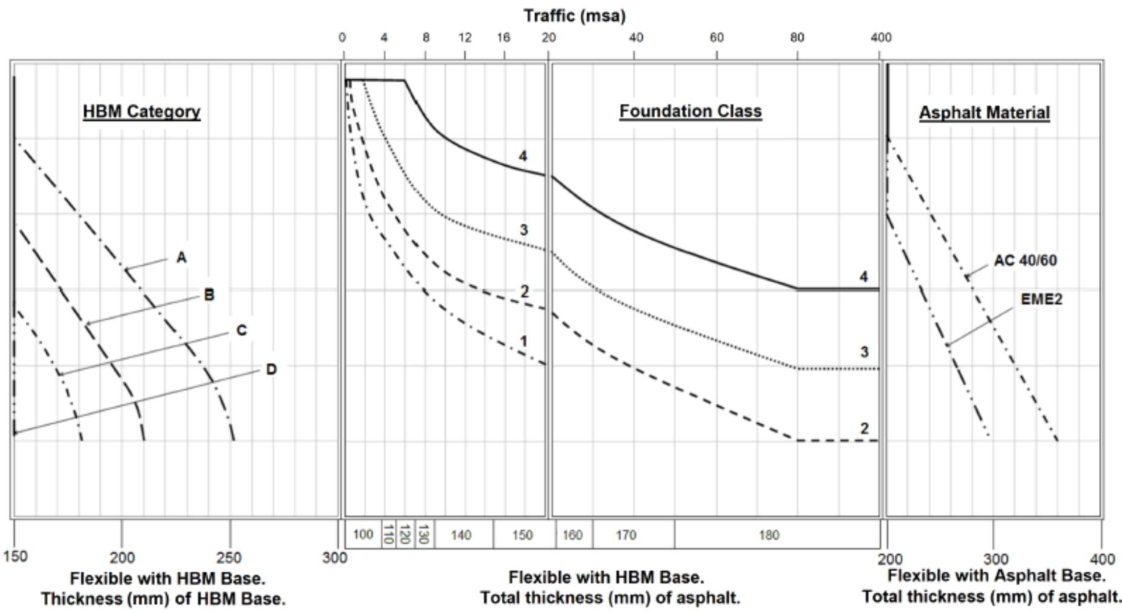
Katendi projekteerimisel tuleb esitada 3 varianti:

- elastne katend asfaldialusel,
- elastne katend hüdruliliselt seotud alusel,
- üks jäik katend.

Uue tee eluiga võetakse 40 aastat, minimaalne normtelgede (80 kN) arv 1 milj.

Elastne katend projekteeritakse vastavalt nomogrammile, lähtutakse liikluskoormusest ja muldkeha klassist. Jäik katend projekteerimine samuti nomogrammide järgi.

Figure 2.20 Nomograph for determining the design thickness for flexible pavements



Tulemusprojekteerimine

Kasutakse analüütilisi meetodeid modelleerides pinget ja deformatsioone ning materjali omadusi kihi paksuste määramiseks (sisuliselt Odemark valem).

Alternatiivse meetodi valik peab olema põhjendatud ja antud täiendavad nõuded materjalile või katsetamisele. Võib kasutada ebastandardseid materjale. Vajalik on vastava organisatsiooni luba. Alternatiivse meetodi printsiibid (elastne ja jäik) on toodud vastavates uurimistöödes.

Drenaaži projekteerimine

CG 501 Design of highway drainage systems. Revision 2. Mar 2020.

Sadevesi hoitakse aluskihist (*sub-base*), tugevduskihist (*capping*) ja aluspinnasest (*sub-grade*) eemal drenaažiga. Nõuded drenaažile on esitatud vastavalt kihtide veejuhtivusele.

Kogutud sadevesi esimese variandina kaalutakse alati immutamist tee kõrval (imbväljakud). Kui see pole võimalik, siis kaalutakse juhtimist pinnaveekogusse (kraavid).Kui ka see pole võimalik, projekteeritakse sadevee juhtimine sadekanalistasiooni või kanalisatsiooni.

Kokkuvõte ja järeldused UK

Aluspinnase kihimoodul määratakse alati geotehniliste pinnaseuurimiste käigus ning lähtutakse sellest. Isegi kui pinnamoodulit esialgselt ainult hinnati (uuringutel kasutati ainult puurimist, välikatseid ei tehta), siis kontrollitakse aluspinnase kihimoodulit esimesel võimalusel (katselõigu ehitamine).

Välja töötatud tüüplahendused on väga lihtsad, olemas on nomogrammid kihi paksuste määratlemiseks. Tüüplahenduste välja töötamiseks on kasutatud 144 katselõigu andmeid, mida analüüsiti vastavalt teooriale. UK tüüplahendusi ei saa Eestis kasutamiseks otse üle võtta, kuna materjalid on spetsifitseeritud.

Samas – Eestis ilmselt ei ole ka võimalik tüüplahendusi välja töötada. Peast, reaalsetele katsetele tuginemata, seda teha ei saa. Seega tuleks teha spetsiaalsed katselõigud väga konkreeste materjaliga ja kriteeriumidega, siis võiks tüüplahendused välja töötada.

USA

AASTHO põhimõtted

AASTHO on empiiriline meetod, mis põhineb suuremahulisel katsel ja katelõikudelt kogutud andmetel (Road Test).

Lähtekoht on tee kasutatavus, mida väljendatakse PSI-na (Pavement Serviceability Index, tee kasutatavusindeks) abil, näitab tee sõidetavust (enamasti tasasus, ka praod). Algne kasutusindeks (PSI) ja selle vähenemine aja jooksul teatud piirini on katendi projekteerimise kriteeriumideks.

Kasutatavuse langus (ΔPSI) toimub liikluse, katendi vananemise ja keskkonnatingimuste (savi pundumine ja külmakerge) mõjul.

Projeketeerimine seob liikluskoormuse katendi ehituse ja toimimisega nii elastse kui jäiga katendi puhul.

Võrrand elastsele katendi jaoks:

$$\begin{aligned}\log_{10}(W_{18}) = {} & Z_R \times S_o + 9\ 36 \times \log_{10}(SN + 1) \\ & - 0\ 20 + \frac{\log_{10}\left[\dfrac{\Delta PSI}{4\ 2 - 1\ 5}\right]}{0\ 40 + \dfrac{1094}{(SN + 1)^{5\ 19}}} \\ & + 2\ 32 \times \log_{10}(M_R) - 8\ 07 \qquad (1\ 2\ 1)\end{aligned}$$

W_{18} normtelgede arv

Z_R standard normaalhälve

S_0 kombineeritud standardviga liikluse ja toimimise prognoosil

ΔPSI kasutatavuse indeksi vähenemine

M_R aluskihi moodul (*resilient modulus* ehk taastuv elastsusmoodul, sisuliselt meie E_{V2})

SN struktuurnumber

Struktuurnumber on abstraktne number, mis väljendab katendi struktuurtugevust tingimustes, mis hõlmavad aluspinnase kandevõime (Mr), liikuluskoormuse, tee kasutatavuse ja keskkonnamõjud.

Struktuurnumber seob kihi omadused ja paksuse ning seda kasutatakse katendi kihtide paksuse arvutamisel.

$SN=a_1D_1+a_2D_2m_2+a_3D_3m_3$

a- kihi koefitsient,

D- kihi paksus,

m- drenaaži koefitsient,

indeksid 1, 2, 3 näitavad katendi kihte.

Kihi koefitsiendid (a_1, a_2...) näitavad ehitatava kihi suhtelist tugevust, Kihi koefitsiendid põhinevad M_R-l, sõltuvad pingeolukorrast ning need tuletatakse vastavalt pingetele (s_1 ja s_2 s_3).

Konstruktsiooni sattunud vesi nii infiltereeunud sademed kui pinnasevesi mõjutavad materjalide omadusi. Veeolusid arvestatakse **drenaažikoefitsiendi** kaudu, mis leitakse sõltuvalt drenaaži kvaliteedist (vee väljavoolu kiirusest) ja ajaperioodi pikkusest (%-des), mille jooksul vastav kiht on veeküllastunud.

Aluspinnase (muldkeha) pinnamoodul M_R on geotehniline sisend, mis tuleb katseliselt määrata. Projekteerimiseks leitakse aasta keskmine väärtus, milles väljenduvad keskkonnatingimuste muutused aasta lõikes. Sarnane UK süsteemiga nii sisult kui määramise viisilt.

Projekteerimise usaldusväärsus näitab, kui tõenäoline on, et tee toimib vastavalt projekteeritule oma kasutusea lõpuni. Selleks on võrrandisse sisse viidud vastavad parameetrid (usaldusväärsuse taseme koefitsient).

Tee kuivendamine

Kaks peamist tee toimimist mõjutavat keskkonnatingimust on temperatuur ja sademed.

Temperatuur mõjutab asfaltbetooni (AB) roomet, temperatuurist tulenevad pinged AB-s, tsementbetooni paisumist-kokkutõmbumist, külmumis-sulamis tsüklite mõju.

Vesi satub tee konstruktisooni filtreerudes läbi katte, pinnaseveest. Kui kohe ei eemaldata, siis vähendab sidumata teralise materjali kandevõimet, vähendab aluskihtide kandevõimet, võib tekkida peene materjali pumpamine alumistest kihtidest pinnalähedastesse kihtidesse.

Sademed, mis infiltreeruvad tee konstruktsiooni mõjutavad materjalide omadusi, üritatakse viia miinimumini, milleks vesi kiiresti välja juhitakse.

Vee puhul kolm võimalust:

1. takistada katendisse tungimist, katkestades pinnasevee voolu või tehes katte võimalikult veetihedaks
2. juhtida võimalikult kiiresti välja,

3. projekteerida katend nii tugev, et see peab vastu koormuse ja vee mõjule.

Drenaažikiht 300...1000 m/ööp.

Prognoositakse aega, mille jooksul 50% tee konstruktsiooni sattunud veest voolab välja.

The measurement of subsurface drainage is generally based on the time required for 50-percent of the unbound water to be removed from the layer to be drained The Casagrande flow equation for estimating the 50-percent drainage time is expressed as

$$t_{50} = (\eta_e \times L^2)/[2 \times K \times (H + L \times \tan \alpha)] \quad (1\ 6\ 1)$$

where

t_{50} = time for 50 percent of unbound water to drain (days),
η_e = effective porosity (80 percent of absolute porosity),
L = length of flow path (feet),
K = permeability constant (ft /day), and
tan α = slope of the base layer

Kaks kriteeriumit drenaaži projekteerimiseks:

- aeg, mille jooksul vesi katendist välja voolab (algab üleujutusega ja lõpeb määratletud tasemega).
- juurdevoolu-äravoolu kriteerium- väljavool on võrdne või suurem kui juurdevool

Vee eemaldus ka vertikaalselt sügavamale, (sel juhul peab alumise kihi filtratsioonimoodul olema suurem kui ülemise kihi oma), mööda vett juhtivat kihti lateraalselt vee kogumis ja ärajuhtimissüsteemi või mõlemad kombinatsioonis.

AASTHO arvestab vett järgmiselt:

Uute teede puhul:

vee mõju väljendub kihi koefitsiendis drenaažiteguri kaudu:

arvestatakse drenaaži kvaliteeti (vee väljavoolu kiirus)

aeg protsentuaalselt, mil kiht on veeküllastunud.

Rekonstrueerimise puhul arvestatakse olemasolevat olukorda.

Projekteerimise usaldusväärsus (reliability) on tõenäosus, et projekteeritud katend toimib liiklus- ja keskkonnatingimustes oma eluea jooksul rahuldavalt. Matemaatilise analüüsiga leitakse vastavad parameetrid, mida rakendatakse projekteerimise võrrandis ja väljendatakse Z_R standard normaalhälve, S_0 kombineeritud standardviga liikluse ja toimimise prognoosil.

Piirangud tulenevad aluseks olnud katsetingimustest ja mahust, mida siiski muude katsetega on püütud korrigeerida.

Puudused

Põhineb katsel, mis tehtud konkreetsetes tingimustes, konkreetsete materjalidega, teatud aja jooksul (2 aastat) Tulemusi ekstrapoleeritakse 10-20 a peale.

Projekteerimise põhikriteerium - kasutatavuse vähenemine aja jooksul on subjektiivne.

Eelised

Arvestab vee mõjuga muldkehas.

Arvestab külmakerke pikaajalist mõju.

Sisse on toodud tõenäosus.

Lähtub katendi aluskihi reaalsest määratud M_R (*resilient modulus*) väärtusest.

Mehaanilis-empiiriline mudel

Põhineb eeldusel, et katendit saab käsitleda mitmekihilise elastse või visko-elastse struktuurina elastsel või visko-elastsel aluspinnasel. Saab arvutada pinged ja deformatsioonid igas punktis. Sisuliselt on tegemist jälle Odemark valemiga.

Arvutatakse välise koormuse, temperatuuri ja niiskuse mõjul mitmekihilises süsteemis (katendis) tekkivad pinged ja deformatsioonid või paindumine (*deflection*). Tõlgendab analüütilise arvutuse katendi reaktsioonina, milleks on kahjustused – praod, augud, roopad jms.

Siiski mõjutavad katendit ka muud tegurid ning selle tõttu tuleb mudel kalibreerida empiiriliste toimeandmete järgi.

Eelised

Aluseks on korrektne teooria, mistõttu on väljund usaldusväärsem.

Paindlikum ja odavam tee, võimaldab prognoosida konkreetset kahjustust, praod, roopad, võimaldab modelleerida olukordi juhul kui, mis empiiriliselt pole võimalikud.

Puudused

Võimaldaks täpsemalt tee käitumist prognoosida aga reaalseid andmeid on vähe.

Kalibreerimine keeruline, vaja koolitada projekteerijaid, kes harjunud tüüplahendustega.

VENEMAA

N Vene (BCH 46-83) ja VF (ODN 218.046.01) süsteem

Põhimõte

Vene süsteem on **mehaanilis-empiiriline**. Arvutatakse väliskoormuse, temperatuuri ja niiskuse mõjul mitmekihilises süsteemis (katendis) tekkivad pinged ja deformatsioonid või paindumine (deflection).

Põhineb eeldusel, et katendit saab käsitleda mitmekihilise elastse või visko-elastse struktuurina elastsel või visko-elastsel aluspinnasel. saab arvutada pinged ja deformatsioonid igas punktis (sisuliselt Odemark valem).

Teiselt poolt leitakse liikluskoormusele ja elueale vastavalt sihtväärtuse tee kandevõimele (sisuliselt elastsusmoodulile katte/asfaldi/tee pinnal). Tee projekteeritakse selle eesmärgi saavutamiseks. Ühtlasi kontrollitakse nihkepingete lubatavust erinevates kihtides, külmakergete suurust võrreldes neid lubatuga (ette antud vastavalt tee klassile) ja lahendatakse eelkõige sulamisperioodil liigse vee välja juhtimine konstruktsioonist.

Võrdlus teiste süsteemidega – arvutusvalemid ja arvutamispõhimõtted

Katendi elastsel poolruumil põhinevad arvutusvalemid (kandevõime kontroll – elastsusmoodul mitmekihilisel erineva paksuse ja elastsusmooduliga kihilisel aluse pinal, nihkepinged) on sarnased (et mitte ütelda samad – tegelikult on samad sest on Bussinesq pingejaotuse geomeetrilised lahendid, mis leiavad pingetensori komponendid pingeallikast suvalisel kaugusel asuva suvaliselt orienteeritud tensorkuubi tahkudel).

Vee välja juhtimise arvutus põhineb Darci seadusel – veevoolamine veeküllastunud pinnases.

Külmakerkearvutus põhineb pinnaste soojajuhtivusel ja külmaperioodi pikkusel.

Võrdlus teiste süsteemidega – pinnased

Igasugused arvutused saab teha reaalsete numbritega.

Lääne süsteemid pühendavad suurt tähelepanu nende numbrite määramisele, olgu see CBR, M_R, või pinnase tugevusparameetrid (sisehõõrdenurk φ ja nidusus c'). Aluspinnase omadused saadakse uuringute käigus tehtud katsetega sh välikatsetega. Lääne süsteemid nõuavad mehaanilis-empiiriliste süsteemide korral tulemuse kontrollimist ehitamise ajal (katseväljakud).

Ka vene süsteemides saadakse vajalikud numbrilised suurused aluspinnase kohta uuringutega. Kuid uuringute eesmärgiks on pinnaste nimetuse määramine ja nii katsetatakse pinnaseid nimetuse määramise eesmärgil – määratakse lõimis ja savipinnastel plastsusomadused (Atterbergi piirid), sest nende alusel antakse pinnasele nimetus (GOST - 25100). Savipinnaste jaoks on sõltuvalt niiksupiirkonnast sisse viidud nn „parandid", mis arvutatakse igal konkreetsel juhul.

Süsteem on välja töötatud 1/6 planeedi pinnale sobivaks, see on ta arvestab väga erinevaid tingimusi ja tehnilisi võimalusi ning põhineb viimastest kõige primitiivsemal – so puurimisega (aga ka labidaga kaevatud) saadud proovi kvalitatiivsel analüüsil. Pinnase omadusi otse katsetega ei määrata (mitte kunagi), isegi määramise võimalust ette ei nähta. Ehitamise ajal kontrolli empiirilis-mehaanilise mudeli tulemuste osas ei tehta.

Vene süsteemi peamiseks eeliseks on tüüppinnased ja nende tüüpomadused võimaldavad neid käsitleda Mullatööde standardi mõistes pinnaseklassidena.

Konstruktsioonis kasutatavate materjalide omadused on vene süsteemis taas ette antud tabelväärtused ja nende kehtivust ei kontrollita (võrdle UK 144 katselõiku). See põhimõte aga ühtib Mullatööde standardiga – hästi läbi uuritud pinnaste ja materjalide puhul kontrollitakse vaid nende püsiomadusi ja olekuomadusi, funktsionaalseid omadusi viimaste vastavuse kõrrel kontrollida ei ole vajadust.

EESTI

Eesti teede projekteerimine aastatel kuni 1990 ja 1995-tänaseni põhinev vene süsteemil. Eestis sisse viidud täienduste/paranduste/kohanduste ja vene standardite (BCH 46-83, ODN 218.46-01 ja SP 34.133320.2012 koos 2015 aasta parandustega) erisuste võrdlev analüüs on tehtud IPT OÜ poolt 2017 aastal (Töövõtuleping 17-00134/19 „Geotehnikuna tee projekteerimist mõjutavate nõuete ülevaatus ja võrdlus ning eksperthinnagu koostamine koos ettepanekutega nõuete kaasajastamiseks".).

SOOME

Soome teede geotehnilist projekteerimist käsitlev süsteem põhineb algselt Vene süsteemil ja lähtub liikluskoormuse aluse arvutaud vajalikust kandevõimest. Ometi on soomlased oma lahendused edasi arendanud kuni kergesti arusaadavate ja kasutatavate kataloogilahendusteni.

Kataloogilahendused on välja töötatud vastavalt liikluskoormusele ja sõltuvalt aluspinnase omadustest.

LISA 8

ARTIKKEL AVALDAMISEKS TEELEHES

TEE MULDKEHA GEOTEHNILISNE PROJEKTEERIMINE EVS-EN 16907 „MULLATÖÖD“ RAKENDAMISEL
Autor: Peeter Talviste

Autor teostas (kaastöötajatega koos) Transpordiametile uurimistöö kus analüüsisime muldkeha geotehnilise projekteerimise arenguid maailmas ja otsisime Eestile sobivaid toimivaid lahendusi tee muldkeha ja veerežiimi projekteerimisel, ehitamisel ja järelevalvel, mis arvestaksid nii Eesti looduslikke kui inimressursse nii, et oleks tagatud tee konstruktsiooni projekteerimise terviklikkus. Järgnevalt lühike kokkuvõtlik ülevaade Teelehe lugejale, kus selgitame ka laiemale lugejaskonnale muldkeha geotehnilise projekteerimise olemust meil ja mujal ning kus on kattuvused ja üleminekud meie teedeinseneri ja geotehniku pädevustes.

Muldkeha projekteeritakse kõikjal eesmärgiga tagada teekatte pinnal vajalik kandevõime optimaalsete kattekihtide paksustega. Muldkeha erinevate (pinnase- ja/või materjali) kihtide paksus ja järgnevus vajaliku kandevõime saavutamiseks tuletatakse järgmise lahendina:

- Vajalik kandevõime tee pinnale leitakse liikluskoormuse (-sageduse) ja selle prognoosi põhjal. Metoodika erineb riigiti.
- Tee aluspinnase omadused määratakse pinnaseuuringutega, mis ei pea alati tähendama maapinda puuraukude tegemist. Vajadusel (kuluefektiivsema lahenduse saamiseks) looduslike aluspinnaste omadusi parendatakse. Aluspinnaste omaduste leidmise metoodika erineb riigiti.
- Tee konstruktsiooni erinevate kihtide omadused saadakse spetsifikatsioonidest või on leitavad tabelväärtustena. Need omadused tagatakse vajaliku tihendamisega. Nii materjalide kui pinnaste omadused teekonstruktsioonis (spetsifikatsioonide või tabelväärtustena) erinevad riigiti.
- Tee konstruktsioon – erinevate kihtide paksused ja järgnevus – arvutatakse Odemark’i valemi abil. See valem riigiti ei erine ja on elastse poolruumi teoorial põhinev lahend, mis seob koormatud pinna mõõtmed, pingete jaotuse koormatud pinna all ning erineva paksuse ja elastsusega kihtidest koosneva poolruumi reaktsiooni poolruumile rakendatud koormusega.

Lisaks kandevõime tagamisele peab projekteeritud tee konstruktsioon sh muldkeha:

- olema stabiilne, so. ei tohi esile kutsuda maalihet,
- olema ajas püsivate kalletega piki- ja põiksuunas, so. säilitades normi kohased kalded pika aja jooksul,
- olema külmakindel, so. ei tohi kerkida külmumisel rohkem kui lubatud,
- olema stabiilse niiskusrežiimiga, so. välistab hüdrostaatilisest suurema veesurve tekkimise võimaluse, välistab peenosiste materjalide segunemise (nn „ülespumpamise“ teel), välistab materjalide väljakande muldkehast (suffosiooni)

Nii peab tee muldkeha konstruktsiooni geotehniline projekteerimine lahendama järgmised teede pikaealisust tagavad ülesanded:

1. Tee stabiilsus – tee ei põhjusta maalihet aluspinnases ja tee jaoks rajatava süvendi nõlvad on stabiilsed.
2. Tee pinna kalded – tee mõjul toimuv aluspinnase tihenemine ja tihenemise poolt põhjustatud kallete muutus jääb normidega lubatud piiridesse.
3. Tee kandevõime – tee konstruktsioon on projekteeritud pikaealisena vastavalt liikluskoormusele.
4. Tee külmakindlus – tee kerge külmudes jääb lubatud piiridesse.
5. Tee niiskusrežiim – tee konstruktsioonis ei tohi moodustuda ühelgi ajahetkel veeläätsi.
6. Reostunud pinnased – teemaalt uuringutega tuvastatud reostunud pinnase käitlemine.

Nende ülesannete lahendamiseks vajalike andmed saadakse geotehniliste pinnaseuuringute läbi viimisega, mille tulemusel koostatakse pinnasemudel, millest saadava teabe põhjal saab otsustada, mida saab teha.

Geotehniliste pinnaseuuringute eesmärgiks on pinnase (kalju) olukorra kindlaks tegemine ning seda mõjutavate nähtuste iseloomu selgitamine, pinnase ja kalju omaduste määramine ning vajaliku lisateave kogumine muude asjakohaste tingimuste kohta.

Geotehniliste uuringute aruanne peab pinnasemudelis eraldatud geotehniliste üksuste kohta sisaldama vähemalt järgnevate omaduste määranguid:

	Ülesanne	**Geotehniliste kihtide omadused**
1	Tee stabiilsus	Tugevusparameetrid, mahukaal
2	Tee vajumise prognoos ja tee pinna kallete kontroll	Mahumuutusparameetrid, mahukaal
3	Tee kandevõime	Elastsusmoodul, lõimis ja platsus
4	Tee külmakindlus	Lõimis ja platsus
5	Tee niiskusrežiim	Lõimis ja platsus

Eurokood EVS-EN 16907-1 ja 2:2018 kohaselt on võimalik mullatööde, sh tee muldkeha, (Geotehniline) projekteerimine kahe erineva põhimõtte alusel:

- tüüplahendusena;
- erilahendusena.

Järgnevalt selgitame lugejale mis on teede tüüplahendus ja mis erilahendus.

Tüüplahendusena lahendamine tähendab tüüpmaterjalide kasutamist. Tüüpmaterjalid on kindlate püsiomaduste (lõimis, plastsus) alusel rühmadeks liigitatud pinnased või materjalid. Rühma tunnus määratleb materjali kasutamise pinnastäite erinevates osades, aga ka pinnase või materjali geomehaanilised omadused (tugevusparameetrid ja kokkusurutavus). Tüüplahendusena projekteeritud muldkeha ehitusaegne kontroll tehakse materjali vastavuse kontrollimisena püsiomaduste alusel ja vajalike tihedusnõuete (näiteks Proctori tiheduse suhtes kui selle määramine on võimalik) saavutamise kontrollimisena. Tüüplahenduse alavariant on katalooglahendus.

Pinnaste ja materjalide rühmadeks liigitamise põhimõtted on toodud EVS-EN 16907-2:2018. Paljudel riikidel (Austria, Prantsusmaa, Saksamaa, Norra, Hispaania, Rootsi, Suurbritannia) on rahvuslikud, pikaajalisel kogemusel põhinevad liigitamise alused ja need on toodud EVS-EN 16907-1:2018 rahvuslikes lisades. EVS-EN 16907-2:2018 lubab riikidel kasutada dokumendis toodust erinevat, pikaajalisel kogemusel tuginevat rühmadeks jagamist. Samuti on lubatud sobivuse korral kasutada rahvuslikes lisades toodud erinevate riikide siseriiklikke eeskirju. Eesti teede projekteerimise norm ja teedeinseneri kutseõppekavad põhinevad eeltoodul.

Erilahendusena lahendamine võimaldab kasutada pinnaseid ja materjale sõltumata nende rühmast. Sel juhul toimub projekteerimine pinnase ja materjali funktsionaalsete olekuomaduste (tihedus, elastsusmoodul, tugevus) alusel. Erilahenduste kasutamine eeldab katsetöö tegemist, sest katsetöö ajal täpsustatakse muldkehas kasutatavate pinnaste ja materjalide funktsionaalsed omadused ning tihendamise metoodika. Erilahendusena projekteeritud muldkeha ehitusaegne kontroll tehakse saavutatud funktsionaalsete olekuomaduste (sh tee muldkehal elastsusmoodul ehk kandevõime muldkehal jms) kontrollina.

Erilahendustena tee muldkeha projekteerimine on keeruline kuna muutuvad geoloogilised ja hüdrogeoloogilised tingimused ning reljeef eeldavad projekteerimisel muutuvaid kihtide paksuseid. Kihi paksust ei saa aga muuta liiga sageli, eriti arvestades, et tee pinna kõrgus on tavaliselt ette antud suurus. Nii muutuvad pinnaste ja materjalide saavutatavad funktsionaalsed olekuomadused ja nende alusel kihi paksuste optimeerimine sageli vähetähtsaks.

Seetõttu kasutatakse eri riikides valdavalt tüüplahendusi. Tüüplahendused on enamasti välja töötatud kataloogidena, kus olenevalt aluspinnasest, põhjavee veetasemest, muldkehas kasutatud pinnase rühmast ja tee kategooriast on toodud erinevate muldkeha kihtide ja katendi paksused.

Eestis kasutatav elastsete katendite projekteerimise protsess (Elastsete teekatendite projekteerimise juhend (Maanteeameti peadirektori käskkiri nr.0088, 29.03.2017) põhineb algselt dokumendis BCH 46-83 kirjeldatud juhisel. Nimetatud dokumendid klassifitseerivad pinnased vastavalt GOST 25584 pinnaste liigitusele. **EVS-EN 16907 kohaselt liigitab GOST 25584 pinnased püsiomaduste (lõimis, plastsusomadused) alusel rühmadeks**. Rühmadele omistatakse nimetus, tähis ja teedeehituses nõuete kohase tihendamisega saavutatavad olekuomadused: sisehõõrdenurk (F), nidusus (C) ja elastsusmoodul (E). Sarnast klassifitseerimist kasutatakse nii aluspinnase kui muldkeha materjalide puhul. Selline lähenemine on tüüplahendusena projekteerimine, kuid tüüplahendus ei ole vormistatud kataloogideks.

Muldkeha projekteerimine Eestis on tüüplahendusena projekteerimine – kasutatakse püsiomadustel põhinevat rühmadeks jagamist ja rühmade funktsionaalsed omadused (kandevõime, tugevus) on määratud rühma kaudu. Tüüplahendustena projekteerimisel saab Eestis lähtuda KAP arvutusalgoritmiga välja töötatud lahendusest. See lahendus ei ole veel võrreldav eri riikides välja töötatud **katalooglahendustega**. Vene Föderatsioonis on viimastel aastatel vastavad kataloogid tee konstruktsioonile koostatud (PNST 390-2020 ja PNST 542-2021) ja kuigi nende kasutamine tundub olevat kooskõlas Eesti normi olemusega, siis nende kasutamisele peaks eelnema lahenduste analüüs.

Katalooglahendus on tüüplahenduse alaliik, vajalikud arvutused on teostanud katalooglahenduste koostaja ning katalooglahendused haaravad kogu tee konstruktsiooni so. ka katendi lahenduse.

Kui KAP algoritmis (Elastsete teekatendite projekteerimise juhend (Maanteeameti peadirektori käskkiri nr.0088, 29.03.2017) ja Katendiarvutus. Elastsete katendite arvutamise programmi kasutusjuhend. MA 2017-002.) **kasutatakse mõnda teist**, GOST 25584 põhimõtetest erinevat, pinnaste ja materjalide rühmadeks liigitamist on tegemist **erilahendusena** projekteerimisega ja kehtivad erilahenduste projekteerimise põhimõtted ja tulemuse kontrolli võtted EVS-EN 16907-st.

Geosünteetide kasutamise projekteerimine tee konstruktsiooni osana põhineb tootjate tehtud võrdluskatsetel. Sisuliselt on tegemist geosünteetika tootjate poolt loodud **tüüplahendustega**. Geosünteetide kasutamisel koostatud lahenduste juures on oluline, et lahendus kehtib vaid konkreetse tootja geosünteedile mis tehtud konkreetse katendi arvutuse käigus.

Loodusliku aluspinnase omaduste parandamise (**stabiliseerimise**) vajadus võib tekkida seoses muldkeha stabiilsuse tagamisega, muldkeha vajumise vähendamise vajadusega või aluspinnase kandevõime suurendamise vajadusega. Stabiliseerimine on alati erijuhu projekteerimine, kus vastavalt aluspinnase omadustele valitakse vajaliku omaduse tagamiseks otstarbekohane stabiliseeriv aine ja viiakse läbi katsetöö. Stabiliseeritud aluspinnasele võib muldkeha projekteerida nii tüüplahendusena kui erilahendusena. Viimasel juhul peab stabiliseerimise katsetöö sisaldama muldkeha rajamise katsetööd.

Kokkuvõtlikult on tee kandevõime võimalik tagada kolmel erineval moel, mis nõuab vajalike lähteandmete hankimist erinevalt:

- **Kataloogilahenduse** valik vastavalt aluspinnase rühmale ja tee klassile, eri riikide on erinevad kataloogid, mis vastavad aluspinnase rühmale. Liigitus tüüpomadustega pinnaserühmadeks on aluseks ka Soome teede projekteerimisele (Tien geotekninen suunnittelu. 10/2012 Liikenneviraston ohjeita), eksisteerivad vastavad tüüplahenduste kataloogid.
- GOST 25584 alusel klassifitseeritud pinnaserühmade kasutamisel vastavalt „Elastsete teekatendite projekteerimise juhend 2001-52“ (Maanteeamet, 2001) arvutusalgoritmile **tüüplahendusena** arvutamine. Tüüplahendustena arvutatud lahendusi saab kasutada ka Soome pinnaserühmade tüüpomadusi kasutades(Tien geotekninen suunnittelu. 10/2012. Liikenneviraston ohjeita). Enamus riikide projekteerimisnormides selline võimalus puudub.
- Muudel alustel moodustatud pinnaserühmade kasutamisel vastavalt „Elastsete teekatendite projekteerimise juhend“ (Maanteeameti peadirektori käskkiri nr.0088, 29.03.2017) ja „Katendiarvutus. Elastsete katendite arvutamise programmi kasutusjuhend. MA 2017-002.“ arvutusalgoritmile **erilahendusena** arvutamine.

KOKKUVÕTE

Transpordiameti tee projekteerimist käsitlevad juhised ja juhendid ning vastavat tegevust reguleerivad ministri määrused õnnestus ühtlustada läbi töö raames koostatud muldkeha geotehnilise projekteerimise raamistiku Eurokoodeksi põhimõtetega, seda läbi mullatööde standardi EVS-EN 16907 kirjeldatud pinnaseklasside ja neile omistatud tüüpomaduste.

Eestis on aastakümneid katendiarvutuses kasutatud tüüppinnaste tüüpomadusi, kusjuures pinnaseklassidena võib EVS-EN 16907 mõistes lugeda GOST 25584 pinnaseid ja projekteerimise aluseks on neile „Elastsete teekatendite projekteerimise juhendis 2001-52“ omistatud tüüpomadused.

Transpordiamet on aastate vältel arendanud 2001.a. juhendit, täiendades juhendi 2017 aasta versioonis seda mitmete eripinnastega, mille omaduste määramine ei ole vastavuses tüüppinnastele esitatud nõuetega. Selliste pinnaste kasutamine on võimalik ka EVS EN 16907 raamistikus erilahendustena. Erilahenduste projekteerimine on reeglina keerukam, kuid tagab ökonoomsema lahenduse. Erilahenduse teostatavus tuleb enne ehituse algust katsetööga tõestada. Tüüplahenduste ja erilahenduste kvaliteedi kontroll ja töö vastu võtmisel on EVS-EN 16097 kohaselt teatud erisused, millega tuleb projektis arvestada.

Eesti suurusest tulenevalt puudub meil vajalik teadusbaas ja inimressurss oma kataloogilahenduste välja töötamiseks. Kataloogilahendused on kõiki teele esitavaiad nõudeid täitvad läbi arvutatud lahendused tüüppinnastest ja tüüpmaterjalidest ning ei nõua keeruliste insenerarvutuste läbi viimist (arvutused on tehtud kataloogilahenduste välja töötamisel). Käesoleval hetkel puuduvad Eestis sellised kergesti rakendatavad ja kontrollitavad lahendused, ometi on enamus suuremaid riike valdavalt just kataloogilahenduste abil oma teid igapäevaselt projekteerimas. Kindlasti on see üks tulevikusuundi analüüsida heade naabrite abiga missugused katalooglahendused võtta Eestis kasutusse, mis kiirendaks oluliselt projekteerimise protsessi ja tagaks ühtlasemad tingimused kõigile turuosalistele.

LISA 9

Vastused Lisade 1, 2 ja 3 osas tellija detsembris 2021 edastatud Transpordiameti taristu haldamise teenistuse taristu haldamise osakonna küsimustele

Üldised märkused

Vormistada vastavalt varasemalele uuringutele (disclaimer sisekaanele lisada jms).

Esitatu oli eeltutvumise versioon, oleme teadlikud, lisame

Autoriõiguste viide sisekaantel - kontrollida üle kas lepingu järgi ikka tulevad autoriõigused jm TRAM üle.

Vastavalt lepingule on autoriõigused TRAM-le antud. Märkus tagab selle, et muutuste tegemisel IPT koostatud tervikusse ei saa selle muutmisel enam viidata muudetud terviku või teksti muudetud osadele kui IPT tööle.

*Esitada lõpparuanded *.doc failina et saaks aruannete sobivaid osasid juhendite koostmiseks kasutada.*

Aruanded esitatakse ka redigeerimist võimaldava *.doc failina.

Lisa 1 „Tee muldkeha geotehniline projekteerimine“

Uue TPN kavandi (07.06.21) definitsiooni järgi "Muldkeha" ei sisalda katendit (katend on eraldi defineeritud, vt kõrval väljavõte "Uue TPN kavandi definitsioonide väljavõte"). Jääb arusaamatuks miks muldkeha aruandes käsitletakse katendit ning teed kui tervikut läbisegi?

Viidatud on juunikuu kavandile, mida saab ennem vastuvõtmist korrigeerida, arvestades ka määrus 106 viimast eeldatavalt 2021 lõpus jõustuvat muudatust. Selgitame, et ei ole võimalik käsitleda terviku osa sidumata seda tervikuga. Nii tuleb paratamatult vaadelda muldkeha projekteerimist tee kui terviku stabiilsuse, vajumise, kandevõime, külmakindluse ja niiskusrežiimi seisukohalt.

Töö kaas - koostajal puudub teedeala kutsetase. See on vajalik kui tahetakse käsitleda ka katendeid ja teed, mitte ainult muldkeha. Juhul kui ametlikult kaasati vastava kutsetasemega konsultant siis tuleks see oluline info lisada.

Konsultandi kutsetase oli lepingu sõlmimise ajal ja on ka käesoleval hetkel töö tegemiseks piisav.

Uue TPN kavandi ning juurde kuuluva lisaga tuleb katendite käsitlemisel enne kinnitamist kindlasti teha näidisarvutused ja tehniline-majanduslik võrdlus, et teada saada mida uue kontseptsiooni rakendamine endaga reaalselt kaasa toob.

Uus TPN kehtestatakse vastava haldusala ministri poolt alla kirjutatud määrusena. Konsultandil ei ole teada, et MKM kavandaks määrusele lisasid. Konsultant osales eelnõu välja töötamisel geotehnika ala eksperdina tagamaks eelnõu, geotehnika teaduse tänase taseme, sh eelnõu geotehnika standardite ja Eurokoodeksitega ühilduvust.

Kui TPN raamistikus nihke- ja tõmbepingete kriteeriume enam edaspidi ei käsitleta (vaid Odemargi kandevõime kriteeriume), siis läheb nt seniste Soome näidisarvututse järgi (teostatud 2021a volitatud teedeinsener, tase 8 poolt) katendikonstruktsioonid min 30% kallimas.

Käesolev raamistik seob Eurokoodeksiga kõik võimalikud tee konstruktsiooni projekteerimise võimalused.

Esiteks osundab raamistik, et tee konstruktsiooni saab projekteerida tüüpomadustega tüüppinnastele kasutades vastavalt tee klassile valitud kataloogilahendusi. Selle, kõige väiksemat kvalifikatsiooni nõudva lahenduseni, on enamike suuremate riikide juhendid arendatud. Kataloogilahendusi saab omavahel võrrelda vaid koos tee katendiga, milline töö on hetkel töös oleva lepinguga alustatud. Kataloogilahendused tuginevad tüüppinnastele ja tüüpmaterjalidele.

Teiseks saab konstruktsiooni projekteerida tüüppinnaseid ja tüüpmaterjale kasutades. Viimaste omadused on piisava uuritusega ja moodustavad pinnaste ning materjalide klasside kogumiku mullatööde sh. tee muldkeha projekteerimiseks. Raamistik kehtestab tüüppinnaste klassidena „Elastsete teekatendite projekteerimise juhend 2001-52“ toodud pinnased. Seega säilib võimalus loodud tarkvara ja pikaajalise kogemuse kasutamiseks teede projekteerimisel ja ehitamisel.

Kolmandaks saab konstruktsiooni projekteerida ka mitte tüüppinnaseid ja eelkõige nende tüüpomadusi kasutades. „Elastsete teekatendite projekteerimise juhendi“ 2017.a. versioonis (Maanteeameti peadirektori käskkiri nr.0088, 29.03.2017) ja selle kasutusjuhendis on pinnaseid ja materjale, mida ei saa käsitleda Eurokoodi mõistes tüüppinnastena ja nii ei ole neile antud tüüpomadusi projekteerimiseks. Eurostandardi mõistes on tegemist erilahendusega, mis vajab katselõigul kontrolli.

Tüüp-, kataloogi- ja erilahendustele kehtivad erinevad töö kontrolli ja vastu võtmise põhimõtted.

Käesoleva raamistiku tegemiseks on vaja sisulisi geotehnilisi teadmisi, mis katavad kogu tee konstruktsiooni projekteerimise tehnilised alused. Autorite ülesandeks ei ole olnud läbi viia 8 taseme teedeinseneri oskusi nõudvaid kuluanalüüse, mis ei puutu töö ülesandesse, vaid on käsitlenud teede geotehnilist projekteerimist ja sellega kaasnevaid protseduure Eurokoodeksi nõuetele vastava süsteemsusega. Insener tehniliste nõuete sätestamist, tagamaks konstruktsiooni püsivust jm nõuetena ettenähtud aja jooksul, ei saa ega tohi käsitleda ülesandena, kui kalliks see läheb, vaid tehniliste nõuete alusel saab otsustada, missugusesse kohta missugustel tingimustel saab missugused määratud elueaga konstruktsiooni kavandada.

Kindlasti ei ole Soome tüüp- ja kataloogilahendused koostatud nihke- ja tõmbepingete kontrolli läbi viimata.

Kandevõime arvutusteks vajalik Odemarki valem puudub, koos sinna juurde kuuluvate tugevusvaru jms teguritega ning arvutuseekirjadega (igas riigis erinevad nö koolkonnad). Need on kindlasti vajalikud et projekteerija saaks konkreetselt katendikonstruktsiooni arvutada.

Raamistik on muldkeha geotehnilise projekteerimise Eurokoodeksi süsteemi sobitamiseks või viimiseks ja muldkeha ei saa käsitleda katendist lahus (vt. eelmine vastus).

Selgitame, et Odemarki lahend on tahke keha füüsikal põhinev geomeetriline lahend kahe erineva elastsusmooduliga materjali pinnale asetatud koormuse mõjul toimuva deformatsiooni arvutamiseks ning nagu eelnevalt (ja järgnevalt) selgitatud, ei ole raamistiku ülesanne sätestada katendiarvutuse valemeid ega selle tegureid.

Teekatendi projekteerimisel kasutatakse lahendit tee konstruktsiooni pinna vajaliku elastsusmooduli (nimetatakse ka kandevõimeks) leidmiseks. Sisendiks on riigiti erinevad koormused (tingautod).

Eelmise küsimuse vastus – käesoleva raamistik seob praeguse TRAM poolt sätestatud projekteerimise ja ehitamise tegevuse Eurokoodeksi süsteemiga, mis oligi lepingu üheks peamiseks eesmärgiks.

Aruandes on viidatud tüüp- ja kataloogilahendustele - ei leia ühtegi näidist mille järgi saaks hinnangut anda kas nö tüüp ja katalooglahenduste selline ülesehitus on üldse sobilik.

Tüüplahendused on kõik „Elastsete teekatendite projekteerimise juhend 2001-52“ ja selles toodud tüüppinnaseid kasutades projekteeritud lahendused.

Tüüp ja kataloogilahenduste olemust ja kuidas see on Eestis täna on eespool selgitatud.

Mullatööde jagamine tüüp- (kataloogilahendus on tüüplahenduse erijuht) ja erilahendusteks on Eurokoodeksi põhimõte, sellisena rahvusvaheliste komiteede ja erialaühingute poolt vastu võetud ja rakendamiseks kohustuslik.

Puudu on TPN kavandi tabel 40, Dreenivuse hinnanguline tase - hindamise või arvutamise metoodika, et saaks TPN üldse projekteerimisel kasutada.

Uus TPN kehtestatakse vastava haldusala ministri poolt alla kirjutatud määrusena ja sellest tulenevalt TRAM peab kohandama oma juhiste süsteemi. Antud töö üks osa selle süsteemi kooskõlla viimine nii Eurokoodeksi kui sellega seotud EVS-EN 16907 printsiipidega . Konsultant osales eelnõu välja töötamisel geotehnika ala eksperdina tagamaks eelnõu ja Eurokoodeksite ühilduvust.

Vajalik veejuhtivus arvutatakse, arvutuseeskiri on detailides esitatud „Elastsete teekatendite projekteerimise juhend 2001-52“ ja on vee väljaviimiseks sobiva konstruktsiooni projekteerimiseks igati piisav.

Eurocode 7 varutegurite valikut tuleb kindlasti käsitleda (stabiilsus ja vajumiarvutuste juures jm)

Eurokoodek 7 varutegurite süsteem on kirjeldatud EVS-EN 1997-1:2006 ja ette valmistatavas uues versioonis (CEN/TC 250/SC 7 N1436. Eurocode 7 – Geotechnical design. M515 SC7.PT6 1997-1 Geotechnical design – General rules (PT6) Oct-2020.)

Puudub arvutusteks vajalik liikluskoormus (kN/m2), lisada.

Raamistik ei ole projekteerimise juhendi tasemel dokument, mis sätestab kindlad eeskirjad arvutuste tegemiseks vaid raamistik sidumaks TRAM vastavad juhised tervikuks ja koos uue TPNiga kasutamiseks, lisades viited mida TRAMi senistes juhistes ei ole.

Kui see on TRAM juhendi kavand siis tuleb määrata konkreetne meetod projekteerimiseks (mitte jätta kõik võimalused). Samas see TPN raamistik ei kehti siis teistele teeomanikele - MKM peaks selle koos TPN kehtestama.

Raamistik süstematiseerib Eurokoodeksist lähtuvalt tee projekteerimiseks võimalikud lähenemised ja osundab millised lähenemised on Eestis kasutusel ning millised lähenemised on katmata. Raamistik ei ole järjekordne elastsete teekatendite projekteerimise juhendi kavand vaid põhjendatud süsteemne alus teede projekteerimiseks vastavuses Eurokoodeksi põhimõtetega. TRAM-i pädevusse jääb valikud konkreetse meetodi kehtestamiseks või reeglistikuks mis põhineb lõpptulemuse kontrollil. Arvestada tuleb, et olenevalt valikust eeldab Eurokoodeks erinevat lähenemist projekteerimise ja ehitustegevuse kontrollil ja kvaliteedi tagamisel.

Töö teostaja juhib tähelepanu asjaolule, et kõik suuremad riigid on teede projekteerimise lahendanud kataloogilahenduste tasemeni lubades samal ajal tüüplahenduste ja erilahenduste kasutamist neid ka selgelt eristades ning vastavad juhised tuues. Selleks loob TRAM juhendi eelnõu raamistiku.

Lisa 2 „Pinnasemudel teede projekteerimiseks“

*BIM mudeli jaoks on vaja määratleda ka ühtne failiformaat (kehtivas juhendis *.ags nõue).*

Lisatud.

Kehtivas Geotehniliste uuringute juhise p.1.8 järgselt tuleb laboril määrata lisaks EVS järgsele liigitusele ka pinnase tähis (A…G). Need on olulised tähised kehtiva katendiarvutamise juhendi järgselt KAP2.0 arvutuste sisendinfoks (vt täpsemalt Elastsete katendite arvutamise juhend lk 47 all viidatud TTÜ uuring).

Aruande lisas 3.1 kuni 3.7. esitatud tüüppinnaste klassidena ja klassidele omistatud tüüpomadustena.

2017 aasta Elastsete katendite arvutamise juhendis toodu ei ole käsitletav tüüppinnaste tüüpomadustena ja on sellisena kasutav ainut erilahenduste välja töötamisel

Peale eelpool toodud märkuste sisseviimist võib mõnel objektil katsekasutuseks rakendada, et võrrelda eeliseid kehtiva juhendiga.

Lisa 3 „Geosünteetide kasutamine tee konstruktsioonis"

Lisada geokärje kasutusalad, võimalusel ka metallvõrk jms erilahendused.

Geokärjel puudub Euroopas tootestandard.

Ameeriklastel on (2021 aastal) töös standard ASTM D8269-21 „Standard Guide for the Use of Geocells in Geotechnical and Roadway Projects"

Sellises arendusjärgus uue (ja ilmselt liivpinnastes perspektiivse) lahenduse toomine juhistesse pelgalt tootjate katsetuste alusel on meie arvates ennatlik.

Mainima peab et maailmas on olemas veel mitmeid geosünteetika ala tooteid, mis ei ole Euroopas standardiseeritud, kuid ka neid kõiki saab tellija heakskiidetud tootja lahenduste põhjal kasutada, kui vastav võimalus alternatiivideks TRAM hangetes ette näha.

Aruandes tuleks käsitleda ka Elastsete teekatendite projekteerimise juhend p.11.4 viidatud geosünteetidega seonduvaid standardeid.

Aruandes on esitatud teede geotehnilise projekteerimise seisukohalt olulised aspektid.