ASSESSING LIQUEFACTION POTENTIAL OF SOILS UTILISING IN-SITU TESTING

by

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To my father Ahmad Tony & father in law Said Mustafa,
my mother Kustinah & mother in law Syarifah Nadirah,
my wife Syarifah Mastura,
my son Di Raja Qusayyi Rabbani,
and
my daughter Wan Lubnayya Nabigha.

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ABSTRACT

Liquefaction has caused significant failures and represents a significant problem for the community and geotechnical engineering designers (Pyrah et al., 1998). However, in practice, a single reliable method for assessing the liquefaction potential of soils is not well defined, particularly for aged soils. This is due mainly to the fact that most research has been based on 'clean sand' as the calibration to define the boundary between liquefaction and non liquefaction behaviour. Therefore, a well defined procedure for liquefaction assessment which is applicable to soils of any age is a crucial first step in reducing the risk of substructure failures and mitigating casualties resulting from earthquakes.

The research presented herein is focused on investigating the capability of the cone penetration test (*CPT*) and flat dilatometer test (*DMT*) for liquefaction assessment on natural soils considering soils deposited more than 1100 years ago at Gillman, South Australia. The recommended *CPT* procedure from the 1996 *NCEER* and 1998 *NCEER/NSF* Workshops is employed. In addition, the age correction factor proposed by Hayati et al. (2008) is used to revise the cone resistance ratio (*CRR*) values obtained from the *NCEER/NSF* procedure. The *DMT* procedure is selected as another contender in this liquefaction assessment because some researchers, such as Yu et al. (1997), Sladen (1989) and Marchetti (1999), claimed that the *DMT* is able to capture the ageing effect of the soils.

Extensive study to define the peak ground acceleration for this liquefaction assessment is conducted by using one-dimensional, site-specific ground response analysis (*SHAKE91* and *EERA*). The most recent and significant natural earthquake motions recorded by two separate accelerogram stations are obtained and manipulated to suit the data entry format of the response analysis methods. The soil unit weight and its shear wave velocity are derived from *CPT* and *DMT* data by using several empirical correlations. The results are then applied individually to each procedure.

The critical state approach for liquefaction assessment introduced by Jefferies & Been (2006) is used to verify the assessment of both the *CPT* and *DMT* procedures. The simple critical state parameter test proposed by Santamarina & Cho (2003) is undertaken on 6 soil samples taken from the study site to estimate the in-situ state parameter.

Liquefaction assessment using the *CPT* data incorporating ageing and *DMT* procedures (i.e. Marchetti, 1982 and Monaco et al., 2005) are presented and a comparison between all procedures is carried out. Re-examination using critical state approach is made. In addition, the consequences of the liquefaction in terms of ground settlement are also investigated.

Finally, this study shows that soil ageing increases the ability of soil to resist during the seismic loading. Furthermore, by assuming that the critical state approach represents the true conditions of the study site, the liquefaction assessment method proposed by Marchetti (1982) from *DMT* data provides better prediction than the others.

STATEMENT OF ORIGINALITY

This thesis contains no material which has been accepted for the award of any other degree or diploma in any university, or other tertiary institution and, to the best of my knowledge and belief, contains no material previously published or written by another person, except where due reference has been made in the text.

I give consent to this copy of my thesis, when deposited in the University Library, being made available for loan and photocopying.

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In the name of Allah, the Most Gracious, the Most Merciful. All praise is due to Allah, Lord of all that exists and may His peace and blessing be upon His Prophet and Final Messenger Muhammad (pbuh). I am indebted to Allah Subhanahu Wata'ala for the strength, nourishment and opportunities which I have been blessed with during this period of study.

This document encapsulates the last two years of research I have undertaken at the University of Adelaide, Australia. This period has been a special part of my life that I have thoroughly enjoyed and will always look back on with fond memories. This is direct result of the support, friendship and direction that many people have afforded me. First and foremost, I owe a great deal to my principal supervisor, Dr Mark Jaksa from the University of Adelaide. He has provided a great source of direction and support throughout this research. For this I am forever indebted to him. I would also like to share my appreciation for the support of my co-supervisor, Dr William Kaggwa, also from the University of Adelaide. He has provided invaluable direction regarding the laboratory testing and its application.

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ABBREVIATIONS AND NOTATIONS

$\sigma_m^{'}$	additional radial stress for the simple critical state parameter test
$\sigma_{ci}^{'}$	applied pressure for the simple critical state parameter test
A_t	area inside the transparent tube for the simple critical state parameter test
$\sigma_{3i}^{'}$	correction applied pressure for the simple critical state parameter test
ϕ_{cs}	critical state friction angle for the simple critical state parameter test
d_i	distance for the simple critical state parameter test
$p_i^{'}$	mean principal stress in each measurement for the simple critical state parameter test
h_i	measured water level for the simple critical state parameter test
V_{w0}	reference water volume for the simple critical state parameter test
G_s	specific gravity of the soil
γ_w	unit weight of water
e_i	void ratio
ΔV_i	volume change for the simple critical state parameter test
V_t	volume of the device with soil for the simple critical state parameter test
V_d	volume of the device without soil for the simple critical state parameter test
V_s	volume of the soil for the simple critical state parameter test
V_{sp}	volume of the specimen for the simple critical state parameter test
V_{wi}	water volume for the simple critical state parameter test
h_0	reference water level for the simple critical state parameter test

W_s	soil unit weight for the simple critical state parameter test
κ	impedance ratio
Γ	intersect of the CSL at 1 kPa mean stress pressure
ψ	normalising state parameter
λ	slope of the CSL
ε	axial strain in the specimen
Φ	dilatometer friction angle
ϕ‘	internal friction angle
σ'_m	additional radial stress
σ'_{vo}	effective vertical overburden stresses
ψ_0	in situ state parameter
ρ_r,	density of the bedrock
β_s	soil critical damping ratio
ρ_s	soil density or soil unit weight
ε_{vi}	post-liquefaction volumetric strain for the soil sub-layer *i*
σ_{vo}	total vertical overburden stresses
Δz_i	thickness of the sub-layer *i*; and *j* is the number of soil sub-layers
A	amplification ratio
A/D	Analogue to Digital
a_{max}	peak horizontal acceleration at ground surface
a_o	bedrock peak ground acceleration
a_r	incoming bedrock peak ground acceleration to the upper layer.
AS	Australian Standard
ASTM	American Soil Testing and Material
a_t	surface acceleration
BH	Borehole

BP	Before Present
C_A	strength gain factor and
c_h	dilatometer coefficient of consolidation
c_M	membrane correction
CPT	Cone Penetration Test
C_Q	normalising factor of cone penetration resistance
CRR	cyclic resistance ratio
CRR_K	cyclic resistance ratio corrected for age
CS	critical state parameter
CSL	Critical State Line
CSR	Cyclic Stress Ratio
Cu	coefficient of uniformity
c_u	undrained shear strength
D	initial diameter of the specimen
D_{10}	the grain diameter (in mm) corresponding to 10% passing by weight
D_{50}	the grain diameter (in mm) corresponding to 60% passing by weight
DMT	Dilatometer Test
D_R or Dr	relative density
e	void ratio
E_D	dilatometer dilatometer modulus
EERA	Equivalent-linear Earthquake Response Analysis
EPROM	Erasable Programmable Read Only Memory
FS	Factor of Safety,
f_s	sleeve resistance.
g	acceleration of gravity;
G_0	small strain modulus

GHS	the accelerogram at Government House, Adelaide
H	thickness of soil layer
I/O	Input/Output
I_c	soil behaviour index
I_D	dilatometer material index
IBM	International Business Machine
ISOPT	International Symposium on Penetration Testing
ISSMFE	International Society of Soil Mechanics and Foundation Engineering
k	soil specific coefficient proposed by Jefferies & Been (2006)
K_0	coefficient earth at rest
K_c	correction factor of fines content
K_D	dilatometer horizontal stress index
K_{DR}	factor to correct the effect of aging
k_h	dilatometer coefficient of permeability
LCD	Liquid Crystal Displays
LL	Liquid Limit
m	rigidity specific coefficient proposed by Jefferies & Been (2006)
M	earthquake magnitude.
M	compression modulus of the membrane material
MCC	Modified Chinese Criteria
M_{DMT}	dilatometer vertical drained constrained modulus
MPU	*M*icroprocessor Unit
MSF	Magnitude Scaling Factor
n	exponent that varies with soil type
NCEER	National Center for Earthquake Engineering Research
NSF	National Research Foundation
OCR	Overconsolidation Ratio

p_0	dilatometer corrected first reading
p_1	dilatometer corrected second reading
P_a	1 atm of pressure
PGA	Peak horizontal Ground Acceleration
PGV	Peak Ground Velocity
PL	Plastic Limit
Q	normalised parameter of tip resistance
q_c	field cone penetration resistance measured at the tip.
q_{c1N}	normalised penetration resistance
$q_{\mathrm{c1N,cs}}$,	equivalent clean sand normalized tip resistance,
q_D.	penetration resistance of dilatometer blade
Q_p	dimensionless cone resistance based on mean stress
RAM	Random Access Memory
r_d	stress reduction coefficient/factor
S	ground settlement
SASW	Spectral Analysis of Surface Waves
SPT	Standard Penetration Test
t	age or time since initial soil deposition or last critical disturbance
T	wave transmission
TL	termoluminescence
TUK	the accelerogram at Mt. Osmond, Adelaide
U_0	pre-insertion pore water pressure
USCS	Unified Soil Classification System
V_{s}	shear wave velocity
V_{sr}	shear wave velocity of the bedrock
z	depth below the surface

γ_{soil}	total unit weight of soil
ρ	soil mass density
ψ	in-situ state parameter

p.i: Line 19 should read:
... (1989) and Marchetti (1997), claimed that the *DMT* is able to capture the ageing effect of ...

p.3: Line 15 should read:
... originally by Schnabel et al. (1972) and *EERA* by Bardet et al. (2000). For the ...

p.4: Line 22 should read:
... (Schnabel et al., 1972) and *EERA* (Bardet et al., 2000). The results of the site-specific ground ...

p.10: Last line should read:
... dilative behaviour or a strain hardening response. Although cyclic mobility can occur slightly below the normalising state parameter, ψ is an important parameter to detect liquefaction flow, which often causes disastrous failures on structures (Martin & Lew, 1999).

p.12: Line 12 should read:
... less than approximately 10% clay-size fines (< 0.002 mm), and a liquid limit (*LL*) in the ...

p.12: Line 14 should read:
... with more than approximately 10% clay-size fines and a *LL* more than or equal to 32% are ...

p.13, Table 2.1: The correct table is:

Liquid Limit[1] / **Clay-size Content** [2]	**Liquid Limit**[1]**<32%**	**Liquid Limit**[1] **≥32%**
Clay-size Content [2] <10%	Susceptible	Further Studies Required (Considering plastic non-clay sized grains - such as Mica)
Clay-size Content[2] ≥ 10%	Further Studies Required (Considering non-plastic clay sized grains – such as mine and quarry tailings)	Not Susceptible

Notes:
[1]Liquid limit determined by Casagrande-type percussion apparatus.
[2]Clay-size content defined as grains finer than 0.002 mm.

p.15: Line 16 should read:
... cyclic loading (Glaser & Chung, 1995). Moreover, obtaining high quality undisturbed sandy samples is difficult and costly. Thus, in-situ testing is very useful and usually ...

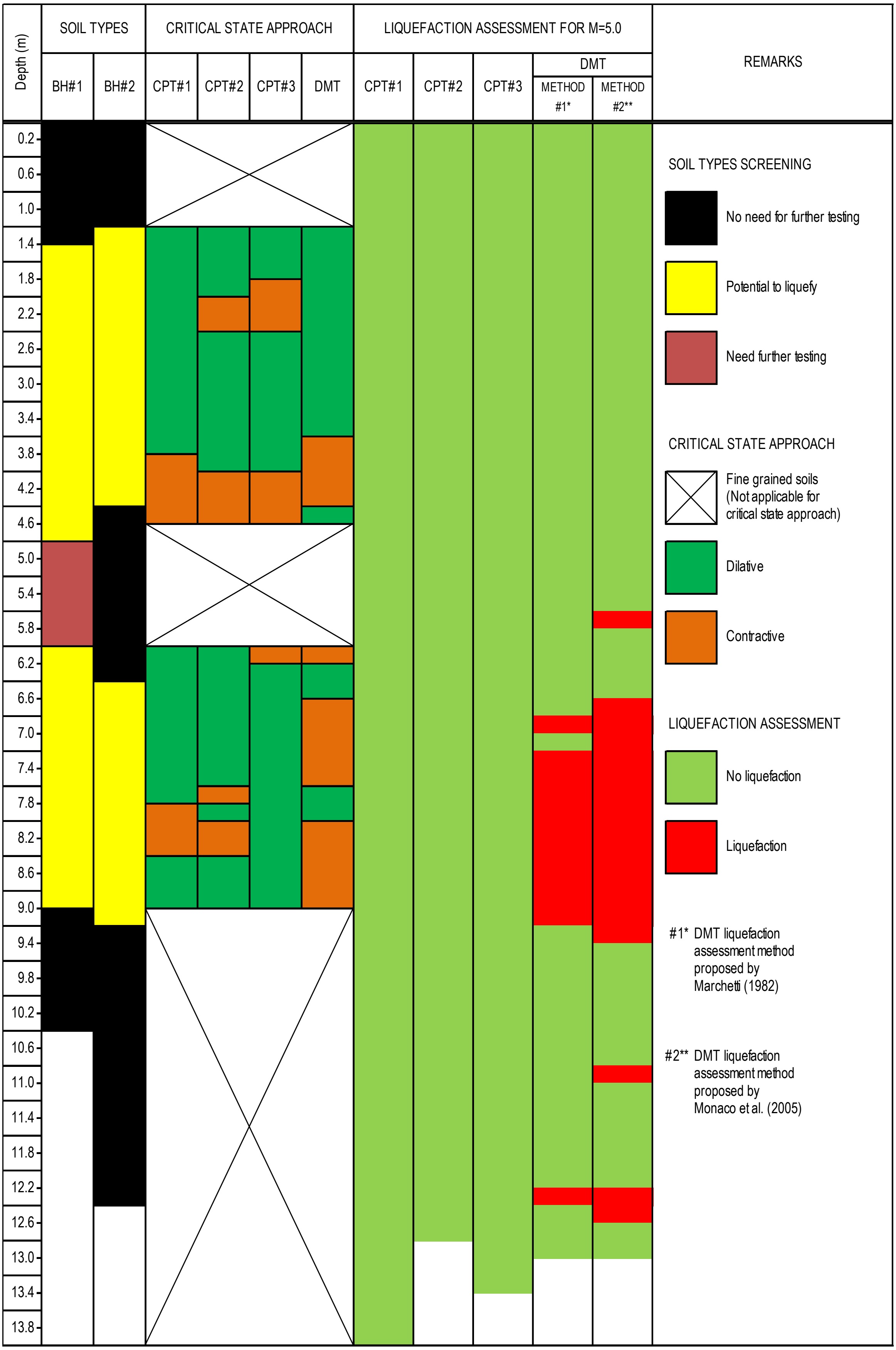
p.175: Line 6 should read:
... 6.4, 6.5 and 6.6 to depict the impact of different magnitude of earthquake at study site. As seen in the plots, the *CSR* values increase when the earthquake...

p.180: Line 2 to 3 should read:
... (identified by green shading) occurs at a depth of approximately 2.2 m; (2) the layer which exhibits the highest impact during an earthquake is the layer at a ...

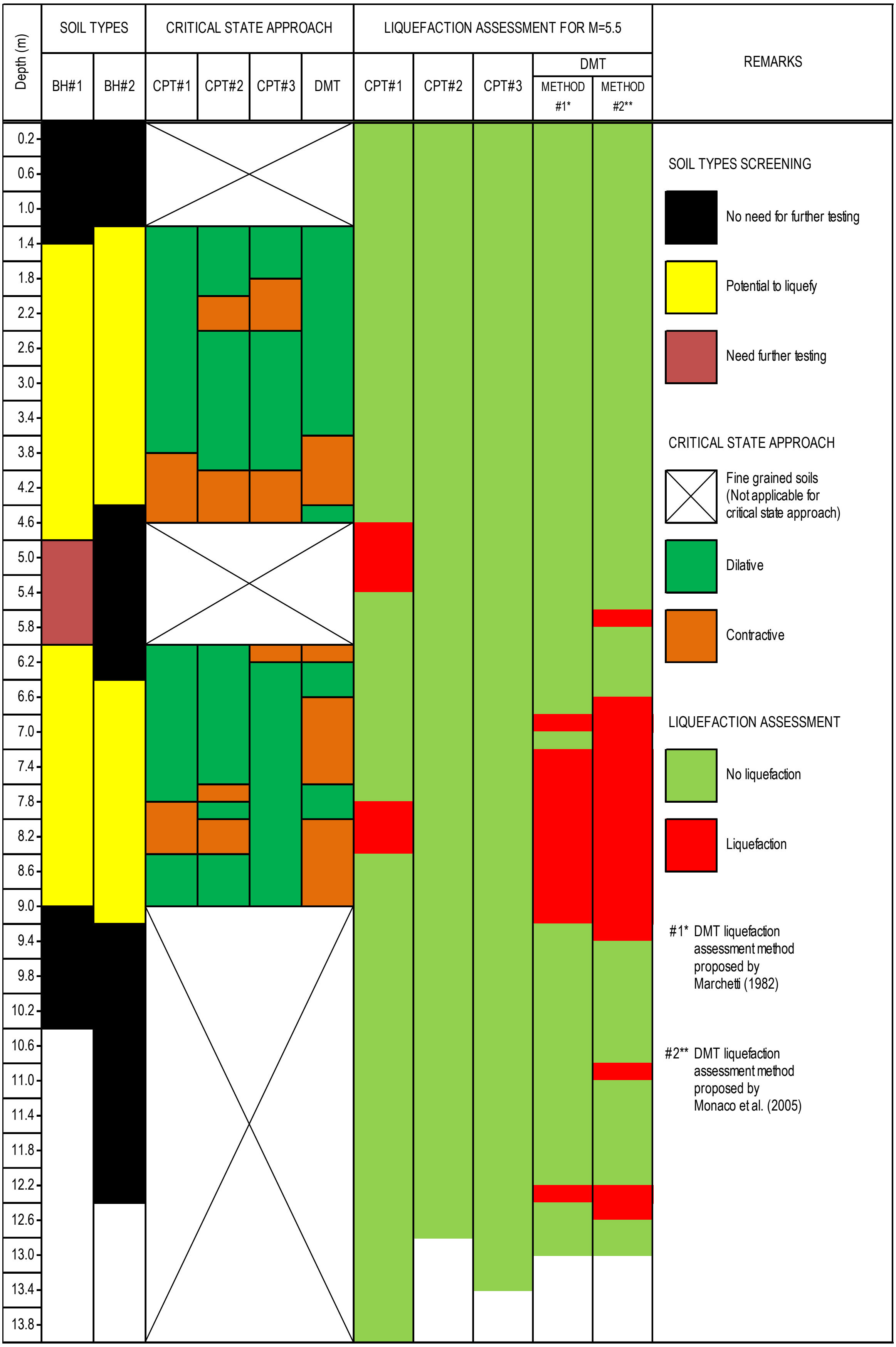
p.180: Line 5 to 6 should read:
... shading); and (3) an increase of 0.5 in earthquake magnitudes causes a rise of the thickness by a factor of 1.5.

p.181: Line 1 should read:
... highest impact during an earthquake is a layer at a depth of between 7.8 to 9.8 m ...

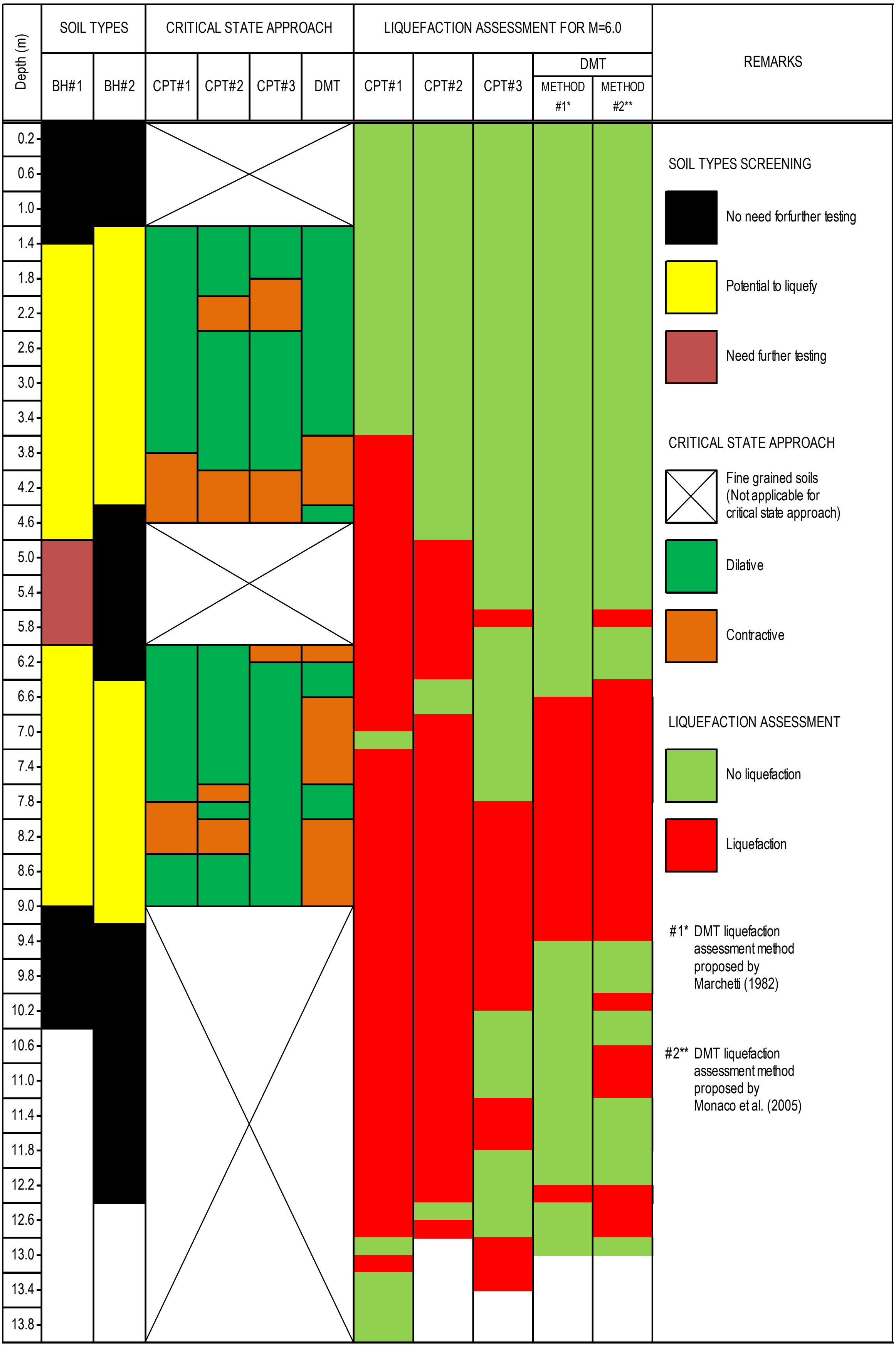
p.202, Table 6.9: The correct table is:



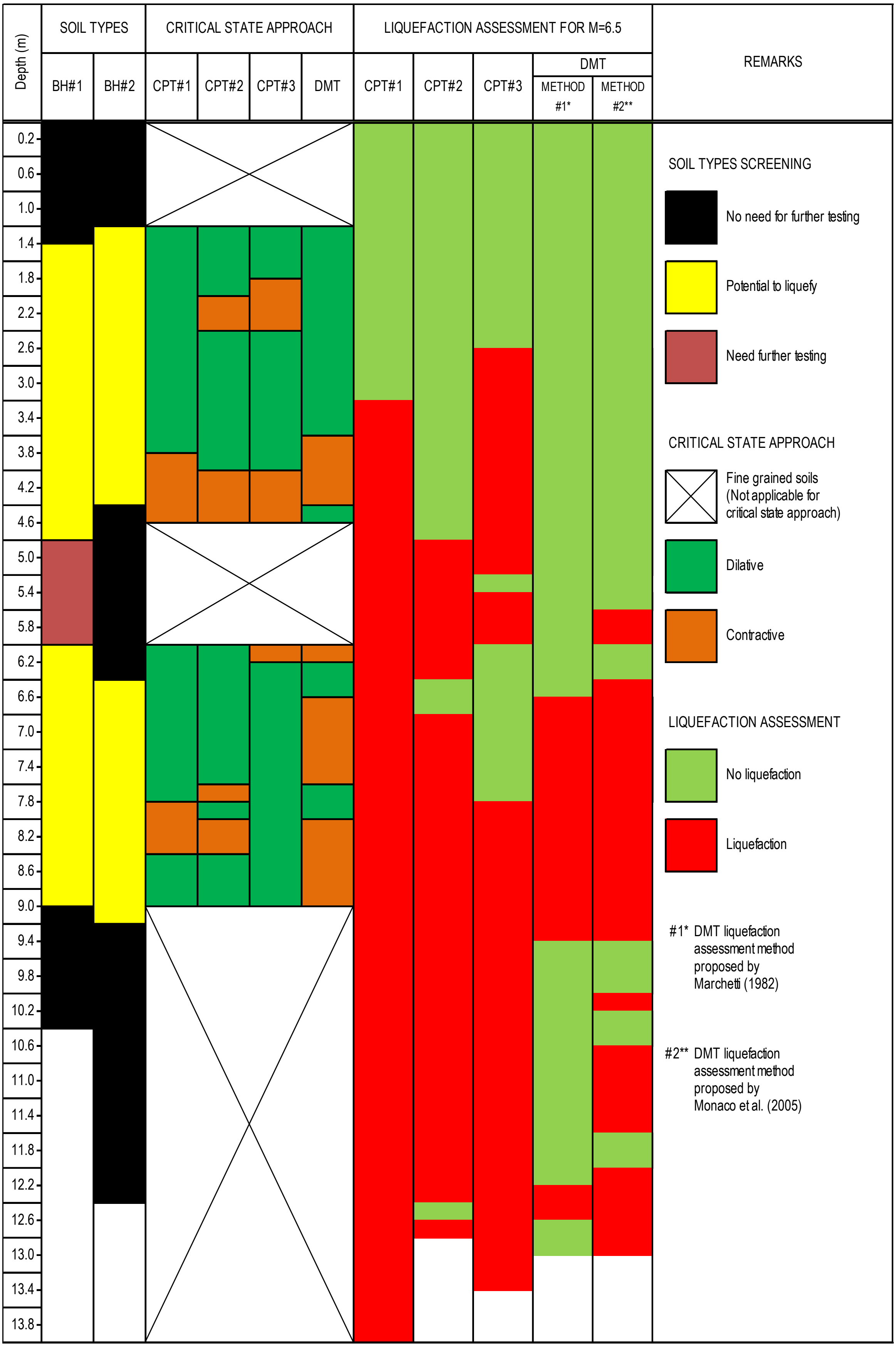
p.203, Table 6.10: The correct table is:



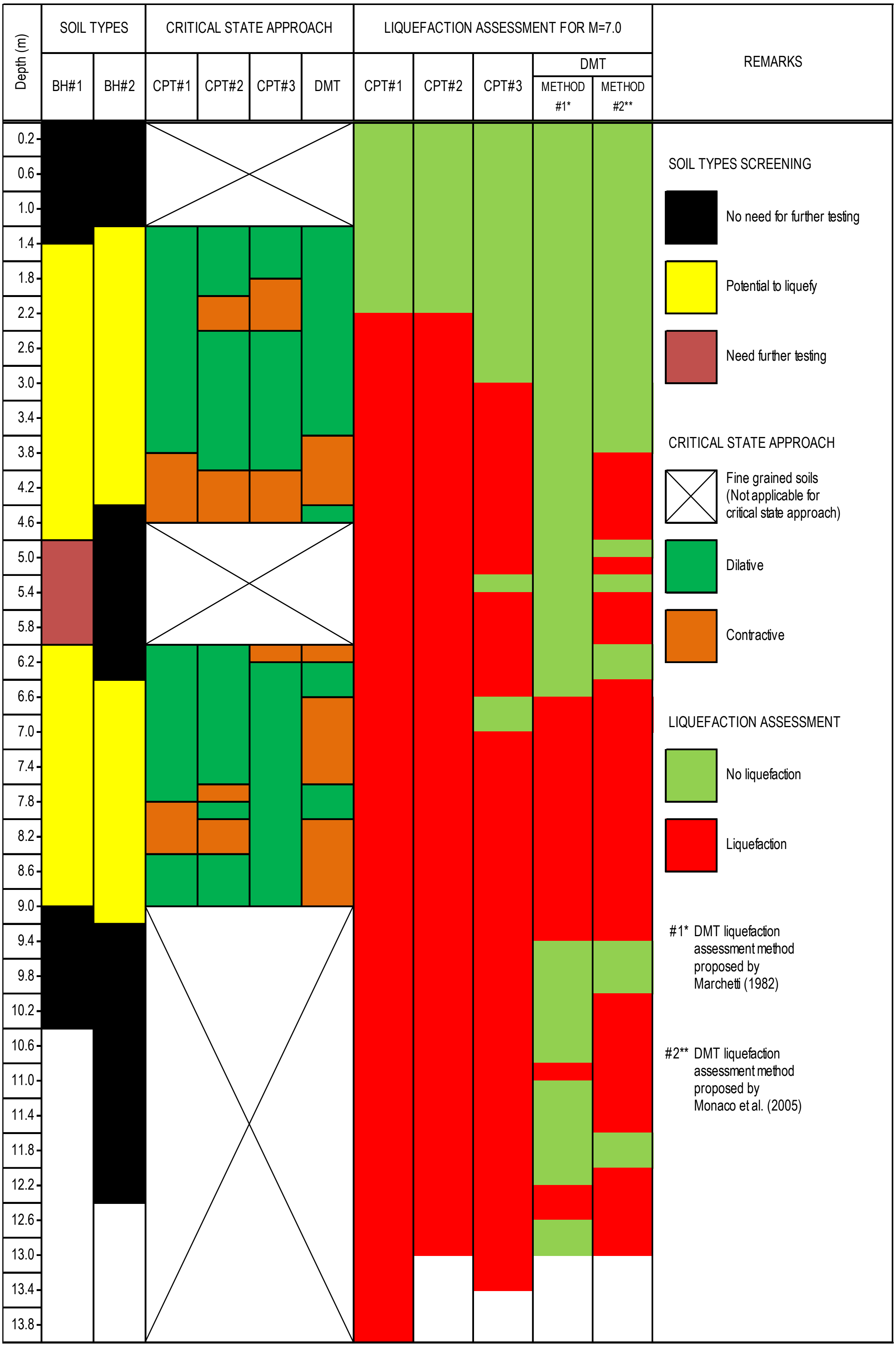
p.204, Table 6.11: The correct table is:



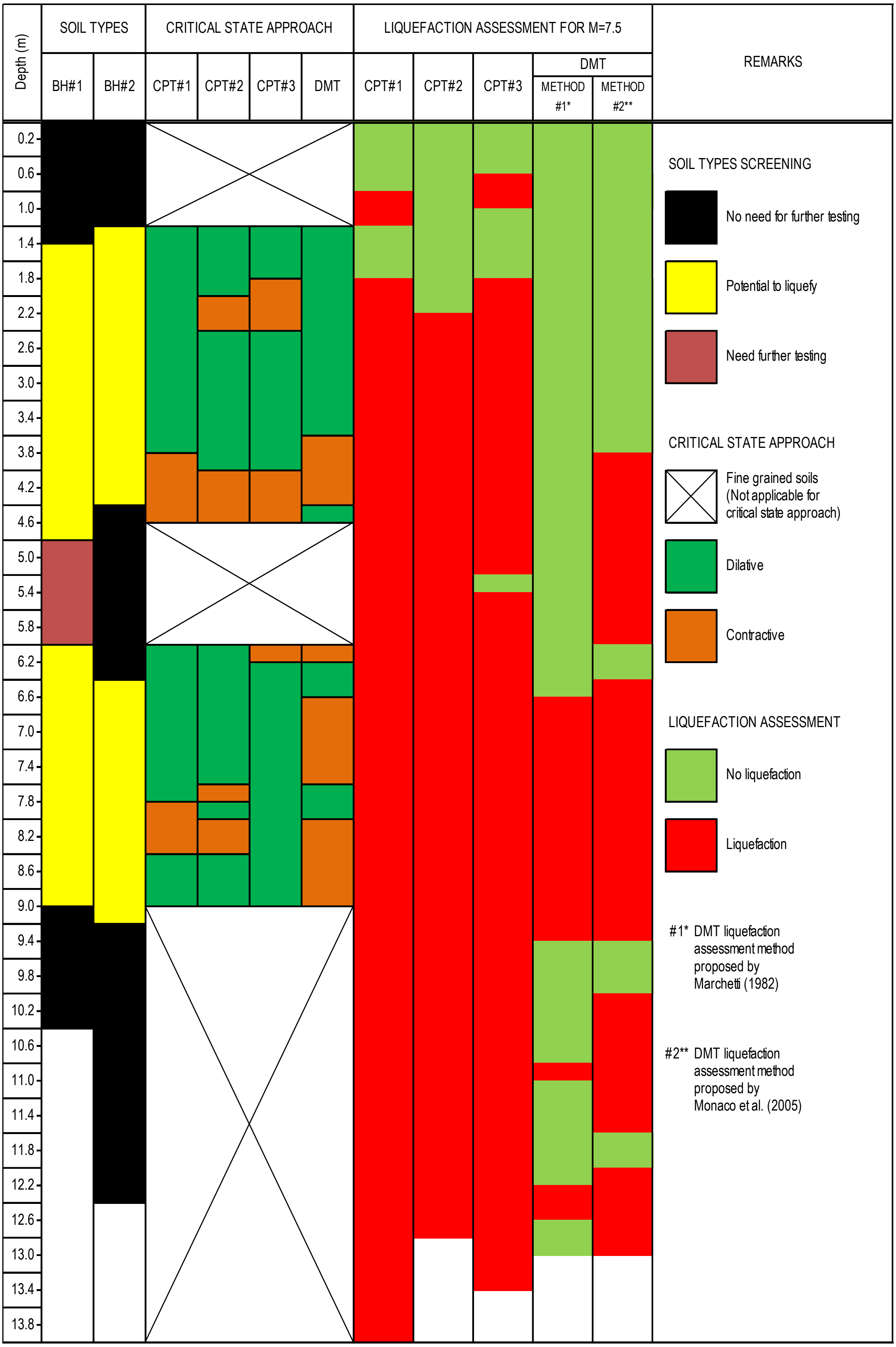
p.205, Table 6.12: The correct table is:



p.206, Table 6.13: The correct table is:



p.207, Table 6.14: The correct table is:



p.216: Line 5 should read:
... specific ground response analysis using the *SHAKE91* (Schnabel et al., 1972) and *EERA*...

p.216: Line 6 should read:
... (Bardet et al., 2000) techniques. The results of the site-specific ground response analysis ...

p.223: Line 28 should read:
... properties of a stiff, overconsolidated clay. Ph.D. Thesis, Faculty of Engineering, The University of Adelaide, Adelaide, 469pp.

p.224: Lines 19 to 20 should read:
... penetrometer tests to estimate settlements of shallow footing on calcareous sand. *Proceedings 7th Australia-New Zealand Conference on Geomechanics,* Adelaide, pp. 909 - 914.

p.227: Lines 15 to 16 should read:
... conventional field testing. *Proceedings 2007 Conference on Earthquake Engineering in Australia*. Australian Earthquake Engineering Society, Paper No. 40, Wollongong, November.