Answers of mid-term exam: Foundation Engineering

1. Table 1 show a geological data on a site. (Ground water table is GL-2m)

(1) Obtain FL values on the liquefiable layers using 1990 version Design Specification for Highway Bridges in Japan. Horizontal seismic coefficient at the ground surface is assumed to be 0.2 considering seismic zone factor and ground condition factor.

(2) Obtain PL value on the ground.

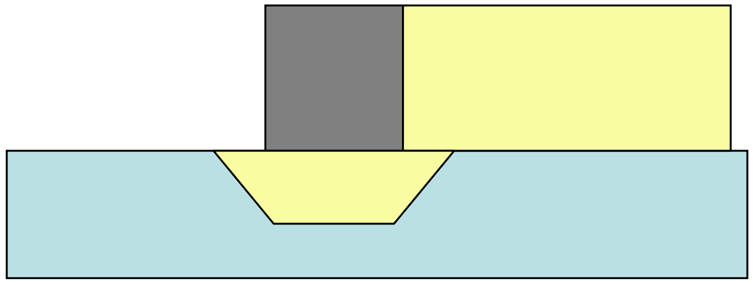
depth (m)	soil type	N valu	γ_{sat} (kgf/cm3	σ_v (kgf/cm2	σ_v' (kgf/cm	FC (%)	D_{50} (mm)	R	r_d	L	FL	ΔPL
1	sand	8	1.9	0.19	0.19	20	0.35	0	0.99	not liquefiable above GW		
2		9	1.9	0.38	0.38	20	0.35	0	0.97			
3		3	1.9	0.57	0.47	20	0.35	0.1412	0.96	0.23164	0.609714	3.32
4		4	1.9	0.76	0.56	20	0.35	0.1571	0.94	0.25514	0.615928	3.07
5		5	1.9	0.95	0.65	10	0.35	0.1697	0.93	0.27038	0.627776	2.79
6		5	1.9	1.14	0.74	10	0.35	0.1644	0.91	0.28038	0.586176	2.90
7		5	1.9	1.33	0.83	10	0.35	0.1594	0.90	0.28683	0.55588	2.89
8		5	1.9	1.52	0.92	10	0.35	0.155	0.88	0.29078	0.532878	2.80
9		5	1.9	1.71	1.01	10	0.35	0.1508	0.87	0.2929	0.514914	2.67
10		5	1.9	1.9	1.1	10	0.35	0.147	0.85	0.29364	0.500619	2.50
11	clay	10	1.8	2.08	1.18	–	0.005		0.84	not liquefiable for clay		PL= 23
12		10	1.8	2.26	1.26	–	0.005		0.82			
13		10	1.8	2.44	1.34	–	0.005		0.81			
14		10	1.8	2.62	1.42	–	0.005		0.79			
15		10	1.8	2.8	1.5	–	0.005		0.78			
16	gravel	40	2	3	1.6	–	10		0.76	not liquefiable for gravel with D_{50} over 2mm		
17		50	2	3.2	1.7	–	10		0.75			
18		50	2	3.4	1.8	–	10		0.73			
19		50	2	3.6	1.9	–	10		0.72			
20		50	2	3.8	2	–	10		0.70			

2. There are many kinds of countermeasure against liquefaction. Discuss suitable countermeasures against soil liquefaction for the following four situations with their principles. You can choose more than one method for each situation.

(1) Quay walls in a newly reclaimed land with very loose sand.

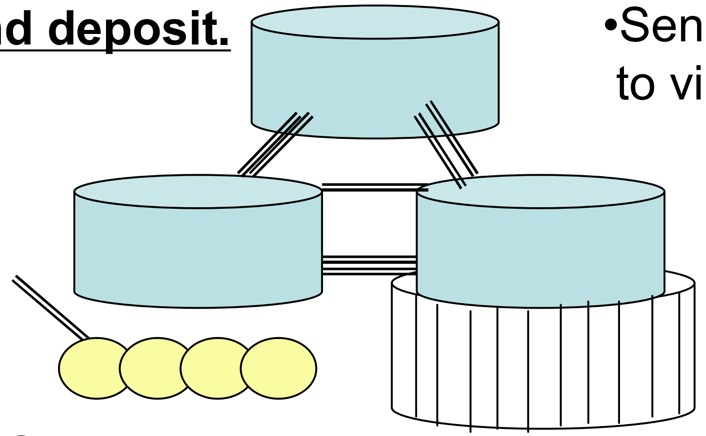
1) For loose reclaimed sand, densification, such as sand compaction, vibro-floatation is the most effective and economical way, if the quay wall can stand the vibration during the pile installation. If not, pore water dissipation methods, such as gravel grain, pipe drain or solidification by DMM or premixing method.

2)For loose foundation soil, if the quay wall can be replaced, densification or replacement are the best way. But if not, solidification by grouting or structural support.



(2) Old tank yard with large number of tanks and pipes founded on a relatively thick loose sand deposit.

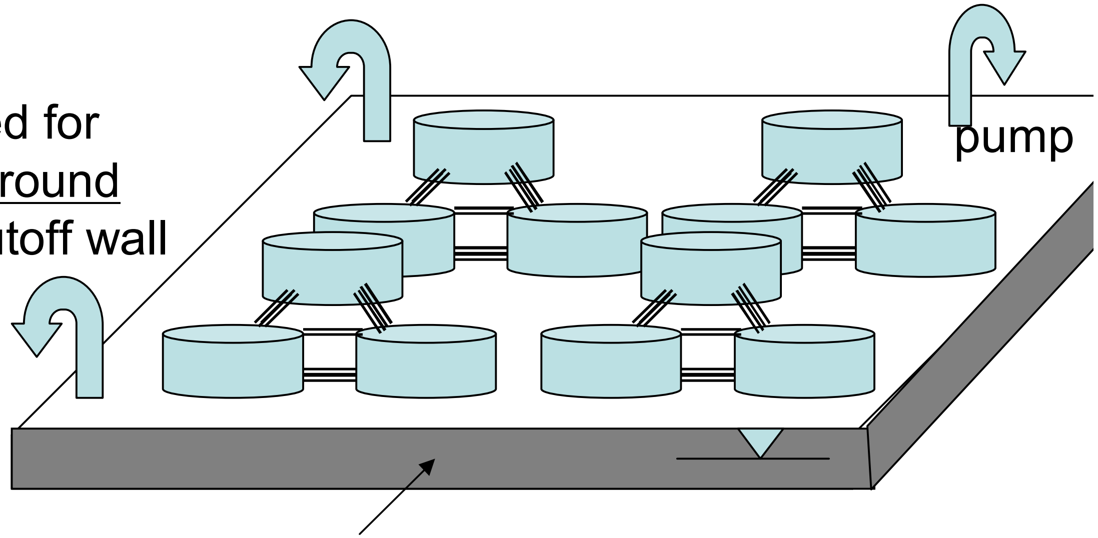
- Narrow Space,
- Sensitive to vibration



Chemical grouting

1) Compaction type methods cannot be applied because of the restriction of hazardous and vulnerable structures (tank and pipes) and small working space. Hence, low vibration and small equipment is preferable, like chemical grouting, sheet pile wall surrounding the tank to constrain the cyclic shear strain and insertion of drainage material for a single tank.

2) If the remediation works are required for whole or large area in the tank yard, ground water lowering by pumping with the cutoff wall surrounding the area can be effective.



Cutoff wall (slurry trench, sheet pile)

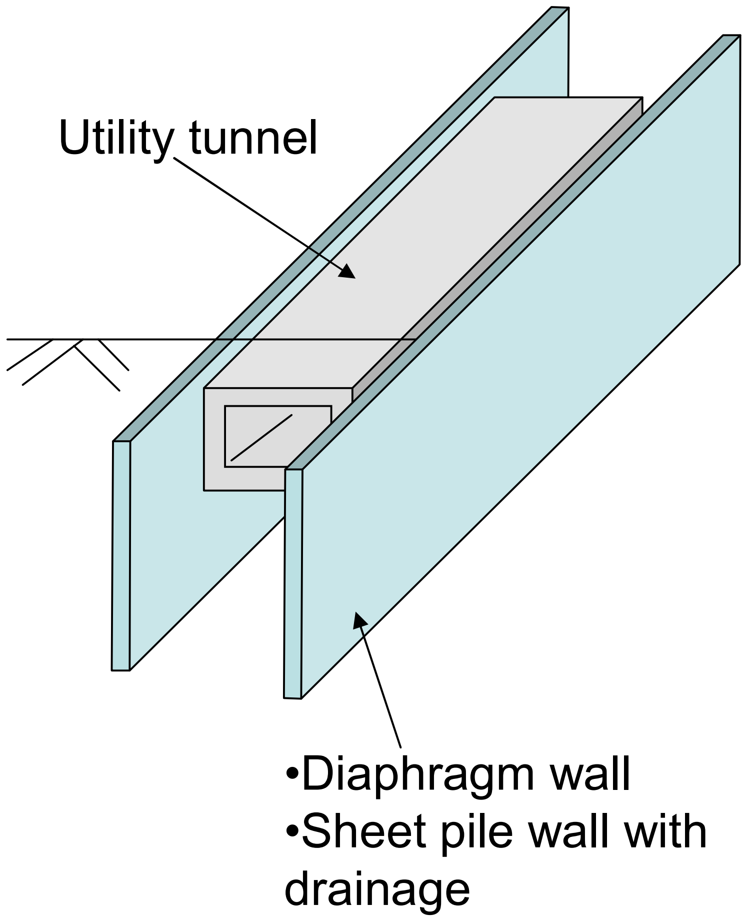
(3) Existing utility tunnel in a liquefiable sand layer.

In case of underground structure, upward movement due to large buoyancy is most common damage due to liquefaction.

Hence dissipation or prevention of excess pore water pressure underneath the structure is effective.

For newly constructed ones, any kind of countermeasures can be applied.

But existing ones, large equipment cannot be used. For this restriction, strain constraint method (e.g., diaphragm wall), dissipation enhancement (e.g., pipe drain) and the combination of these functions (e.g., sheet pile wall with drainage piles) can be applied.



(4)Level ground of loose silty sand with high plasticity (e.g., Fc< 35% but PL>15)

For level ground with less restrictions, such as like noise and vibration, nay kind of remediation methods may be applied. But if the liquefiable soil is silty sand (with high plasticity or high fine contents), it is very difficult to apply the dissipation enhancement method because of low permeability. The efficiency of vibration or dynamic compaction may be less than the pure sand.

For the silty soil, making the soil over-consolidated condition by preloading is one of the effective and economical way.

3. Figure 1 shows a cross section of gravity type quay wall. Using pseudo-static analysis, calculate the caisson width (B) to satisfy the factor of safety (Fs=1.2) on the sliding failure for the following two cases under the given conditions below.

Case 1) No liquefaction occurs so that the vertical effective stresses in the backfill are the same as the ordinary state.

Case 2) Liquefaction occurs and the vertical effective stresses in the backfill sand vanish.

Given conditions

・Height of caisson wall: H=10m;

・Sea water and ground water level: top of caisson;

・Average unit weight of caisson: γ_c=20kN/m^3;

・Saturated unit weight of backfill sand: γ_{sat}=20kN/m^3;

・Unit weight of water: γ_w=10kN/m^3;

・Effective friction angel of backfill sand: ϕ'=30 degree;

・Friction coefficient between caisson and rubble mound: μ=0.7;

・Design seismic coefficient: k_h=0.2;

・Effective active earth pressure $\sigma'_{ha}(z)$ on the wall can be given by

$$\sigma'_{ha}(z) = K_a \sigma'_v(z)$$

where $K_a = \tan^2(45° - \frac{\phi'}{2})$ and $\sigma'_v(z)$ is effective vertical stress.

・Dynamic water pressure is negligible.

Figures 2 & 3 shows the forces and pressures acting on the caisson.
W: total weight of the caisson;
H: horizontal force of self weight of the caisson;
R: resistance against sliding;
p_w: water pressure;
p_a: active earth pressure.

Seismic force, that is body force, is given by (total mass) X (horizontal acceleration: $k_h g$). Hence $H=Wk_h$ ①.
On the other hand, horizontal resistance mobilized at the base by friction is proportional to the effective weight of the caisson. Hence $R= W'\mu$ ②

$W=HB\gamma_c=200B$ kN/m
$W'=HB(\gamma_c-\gamma_w)=100B$ kN/m
$H=40$ kN/m ③
$R=70B$ kN/m ④

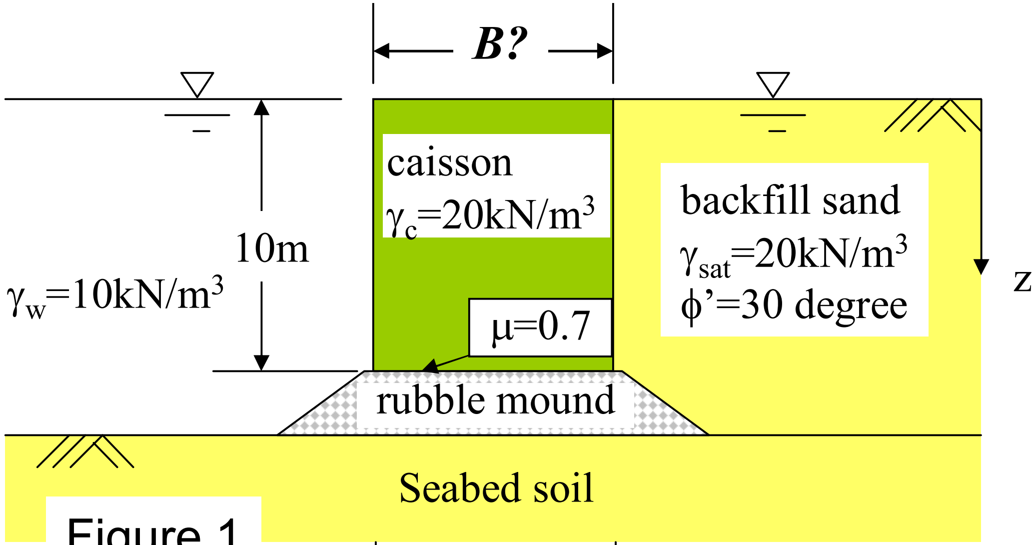


Figure 1

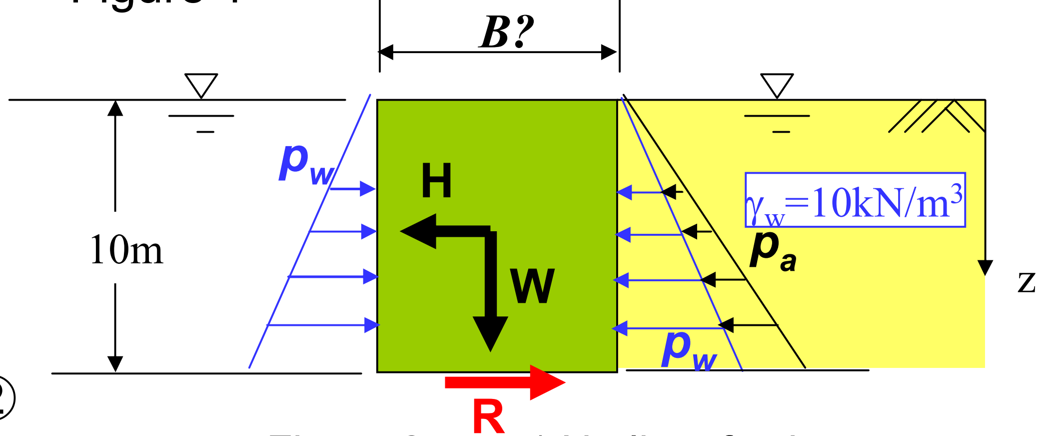


Figure 2 case1:No-liquefaction case

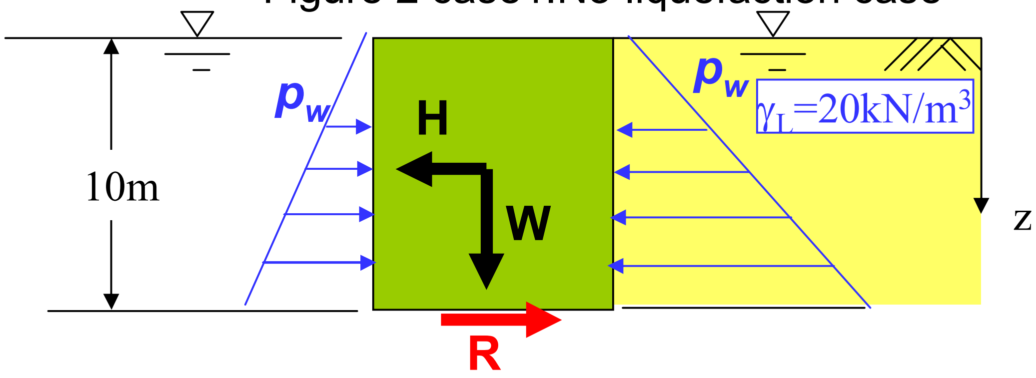


Figure 3 Case2:Liquefaction case

With the assumption that about dynamic water pressure and earth pressure are both negligible, the difference between the cases with and without liquefaction is the horizontal pressure from the backfill side.

In case 1, active earth pressure (p_a) given by Rankine's eq. + static water pressure (p_w) can be assumed. On the other hand, static pressure of a liquid with $\gamma_L=\gamma_{sat}$ can be assumed in case2. Considering static water pressure from the sea side, total horizontal forces by the pressures in case 1 (P1) and in case 2 (P2) are

$$P_1 = K_a \frac{\gamma' H^2}{2} = \frac{500}{3} kN/m, \qquad P_2 = \frac{(\gamma_{sat} - \gamma_w)H^2}{2} = 500 kN/m$$

Fs against sliding is given by the following eq. $F_s = \dfrac{sliding \quad resisntace : R}{total \quad horizontalforece : H + P}$

For case 1: $$F_s = \frac{70B}{40B + 500/3} = 1.2, \qquad hence \quad B = \frac{200}{22} = 9.1m$$

For case 2: $$F_s = \frac{70B}{40B + 500} = 1.2, \qquad hence \quad B = \frac{600}{22} = 27.3m$$

From this example, the significant effect of liquefaction can be confirmed, even for the condition in which no liquefaction at the foundation of the caisson is assumed. The liquefaction of the foundation soil causes significant decrease of horizontal resistance (R), resulting in very low safety factor.