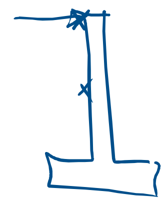
$\underline{\underline{K}}$: Lateral earth pressure factor

① K_o : at rest (no movent allowed in Wall)



$K_o = 1 - \sin\phi$

ϕ → angle friction

② K_a : active earth pressure

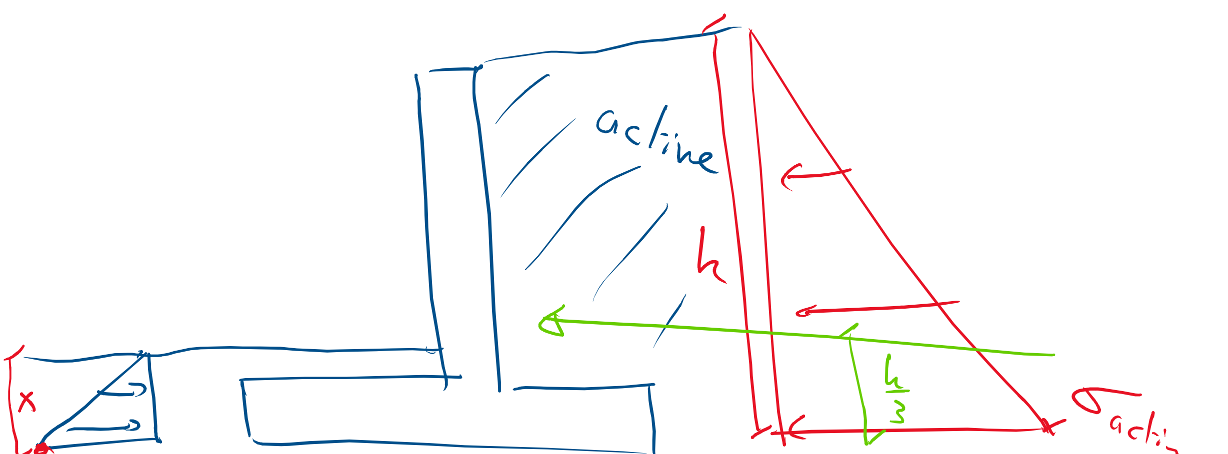
$$K_a = \frac{1 - \sin\phi}{1 + \sin\phi}$$

③ K_p : passive ~~e~~ earth pressure.

$$K_p = \frac{1}{K_a} = \frac{1 + \sin\phi}{1 - \sin\phi}$$

$$K_p > K_a$$

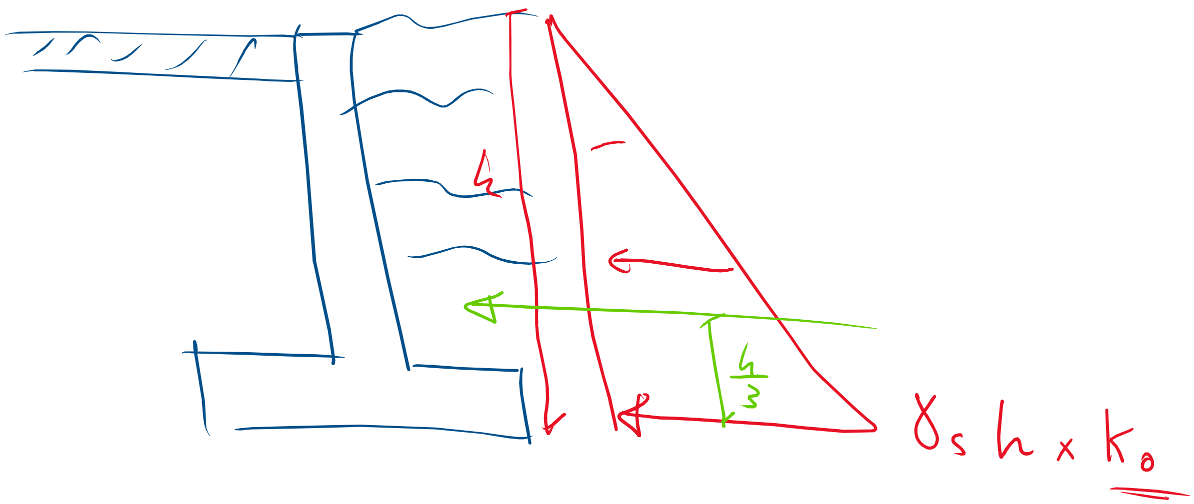
For Cantiliver RW



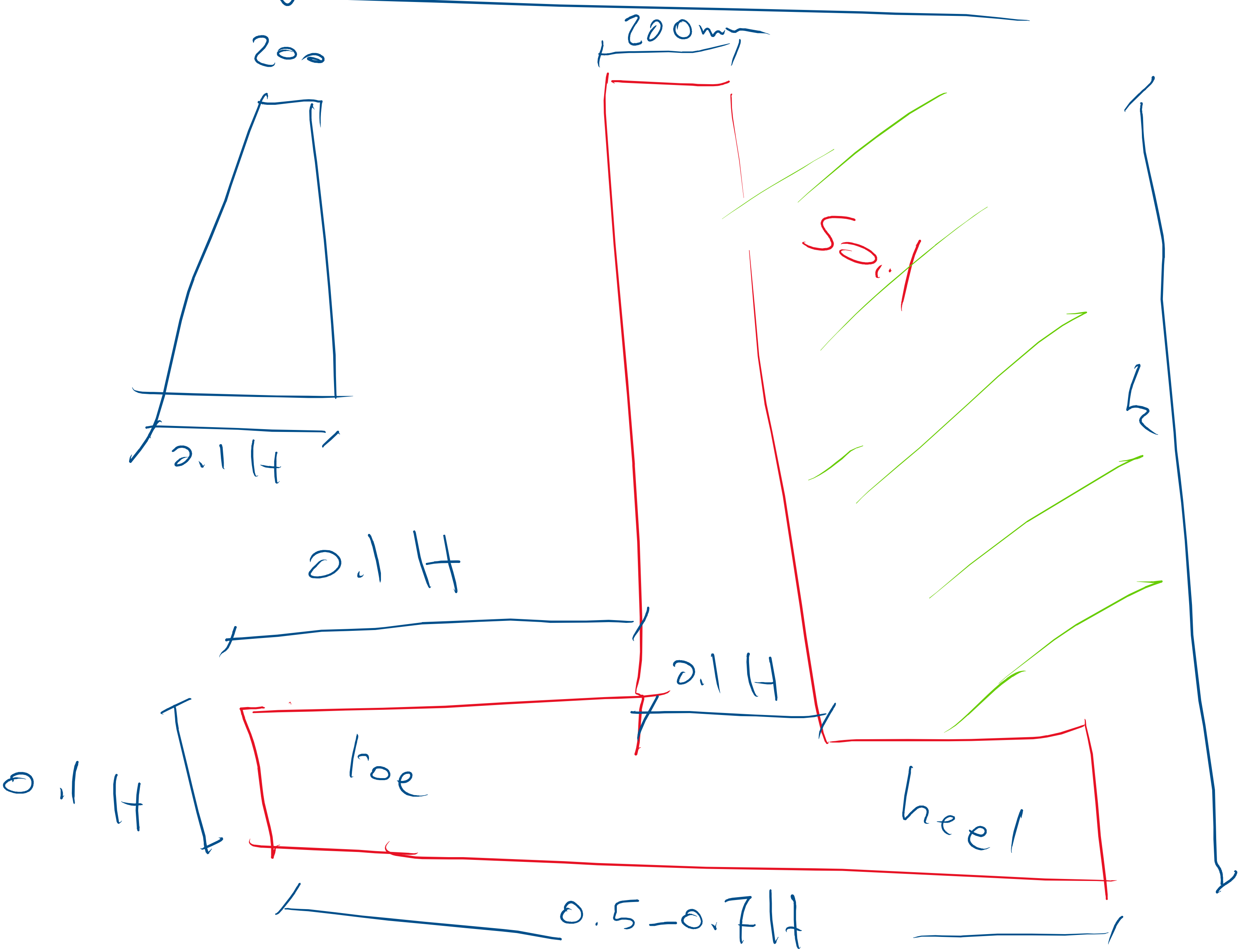
$\sigma_{passive} = \gamma_s . X . K_p$

$\sigma_{active} = \gamma_s h K_a$

For basement Wall



Preliminary Dimension of Cantiliver RW



Example Cantiliver RW

Check and design the Retaining Wall shown Below assuming.

$\phi = 30^\circ$ $\gamma_s = 18\ kN/m^3$

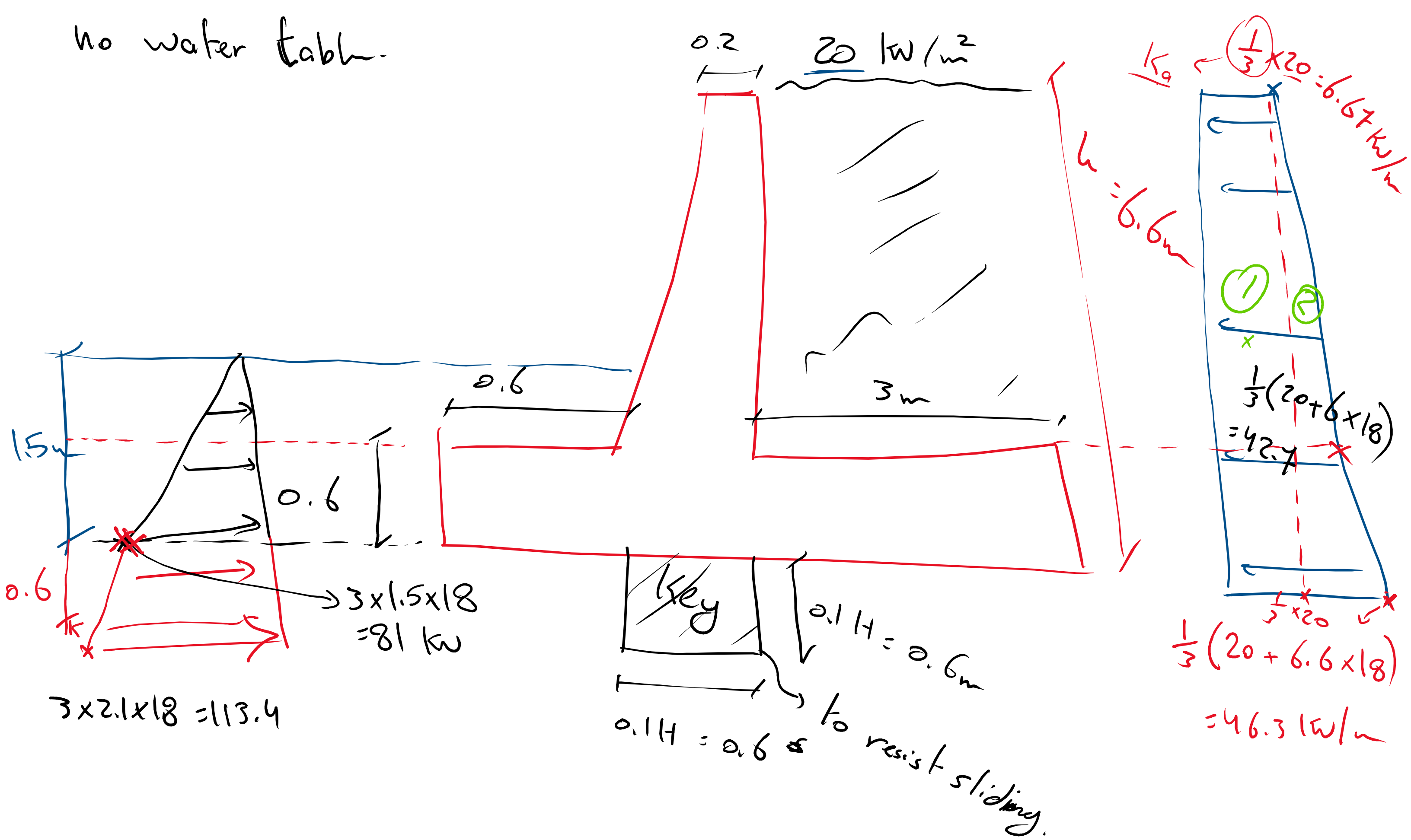
$f'_c = 25\ MPa$. F.S over turning : 2.5 at least

F.S sliding = 2.

$\mu = 0.5$ friction factor ~~Between Wall and Soil~~ (0.3–0.7) larger is Better

$q_{all} = 250\ kN/m^2$

no water table.



Solu

$$K_a = \frac{1 - \sin 30}{1 + \sin 30} = \frac{1}{3} \quad , \quad K_p = \frac{1}{K_a} = 3$$

→ stability → ① over turning √

② sliding √

③ Bearing √

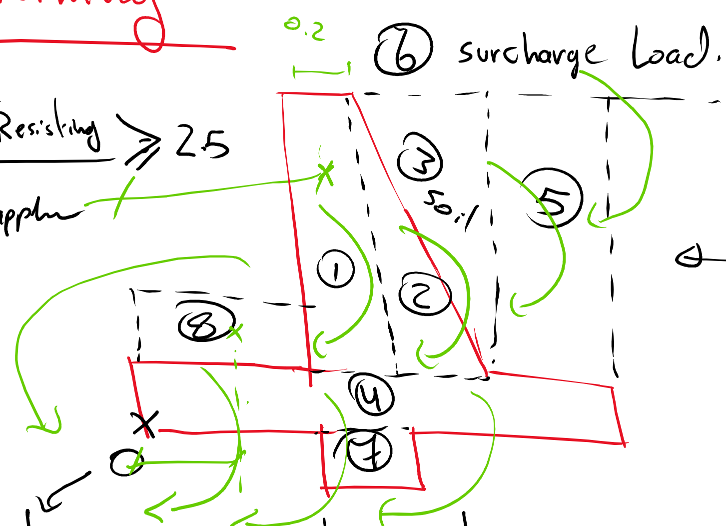
ok

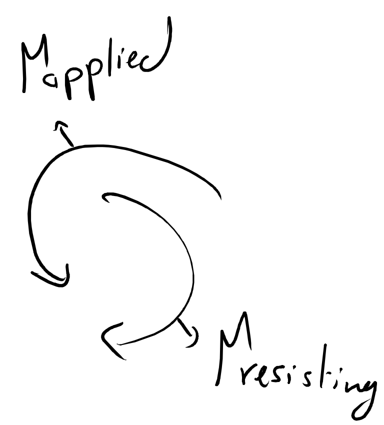
→ strength → ① moment

② shear

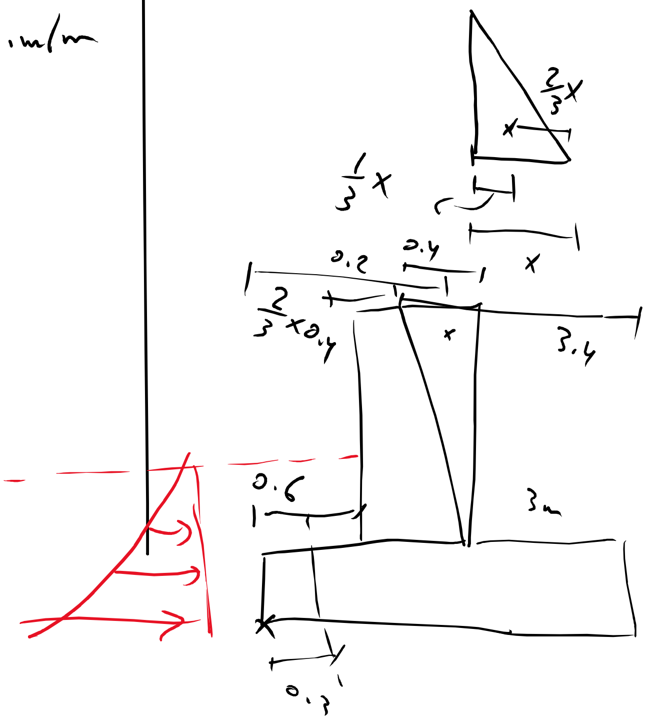
over turning

$$F.S = \frac{M_{Resisting}}{M_{applied}} \geq 2.5$$





Part	Weight (KN/m) (W)	arm (m)	resist moment W x arm
1	$0.2 \times 6 \times \frac{1}{} \times 25 = 30$	$0.6 + \frac{0.2}{2} = 0.7$	21 KN.m/m
2	$\frac{1}{2} \times 0.4 \times 6 \times 25 = 30$	$0.8 + \frac{0.4}{3} = 0.93$	28
3	$\frac{1}{2} \times 0.4 \times 6 \times 18 = 21.6$	$0.8 + \frac{2}{3}(0.4) = 1.07$	23
4	$0.6 \times 4.2 \times 25 = 63$	$\frac{4.2}{2} = 2.1$	132.2
5	$3 \times 6 \times 18 = 324$	$1.2 + \frac{3}{2} = 2.7$	874.8
6	$3.4 \times 20 = 68$	$0.8 + \frac{3.4}{2} = 2.5$	170
7	$0.6 \times 0.6 \times 25 = 9$	$0.6 + \frac{0.6}{2} = 0.9$	8.1
8	$0.9 \times 0.6 \times 18 = 10$	0.3	3
	total = 556 kN/m		$M = 1260$ KN.m/m



Resisting moment $= 1260 + \frac{81 \times 1.5}{2} \times \left(\frac{1.5}{3}\right) = 1290$ kN.m/m

Over turning moment : $M_{ov} = (6.67) \times 6.6 \times \frac{6.6}{2} + (46.3 - 6.67)\left(\frac{6.6}{2}\right)\left(\frac{6.6}{3}\right)$

$= 433$ kN.

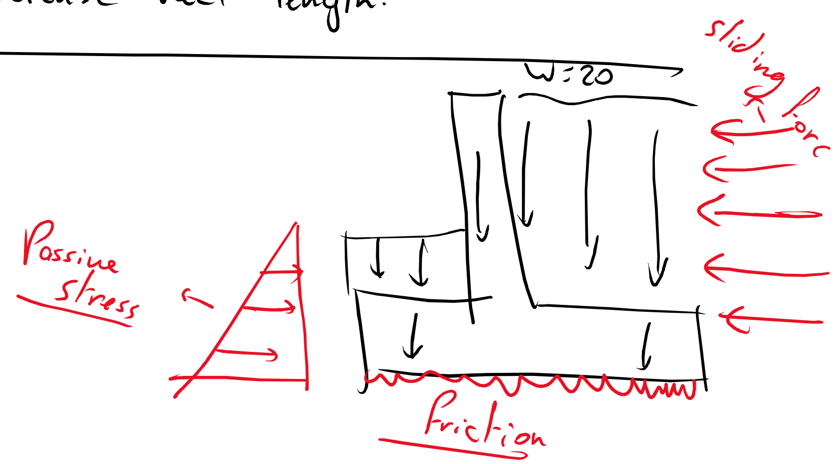
$F.S = \frac{M_R}{M_{OV}} = \frac{1290}{433} = 2.9 > 2.5$ ok

→ can use 1.5 F.S instead.

if F.S Fails in O.T → increase heel length.

Sliding

$F.S \geq 2.0$ ok ?



$$P_{sliding} = \underline{6.67 \times 6.6} + (46.3 - 6.67) \times \frac{6.6}{2} = 174.4 \ kN/m$$

$$P_{resistant} = \underbrace{\frac{1}{2}(113.4) \times 2.1}_{\text{Passive force}} + \underbrace{556 (\overset{\mu}{0.5})}_{\text{Friction}} = 397 \ kN/m$$

$$F.S_{sliding} = \frac{P_{resistant}}{P_{sliding}} = \frac{397}{174.4} = 2.2 > 2 \ \underline{ok}$$

if $F.S_{sliding}$ fails
- ① increase Key Depth → increase passive force.
- ② increase footing width → increase friction.

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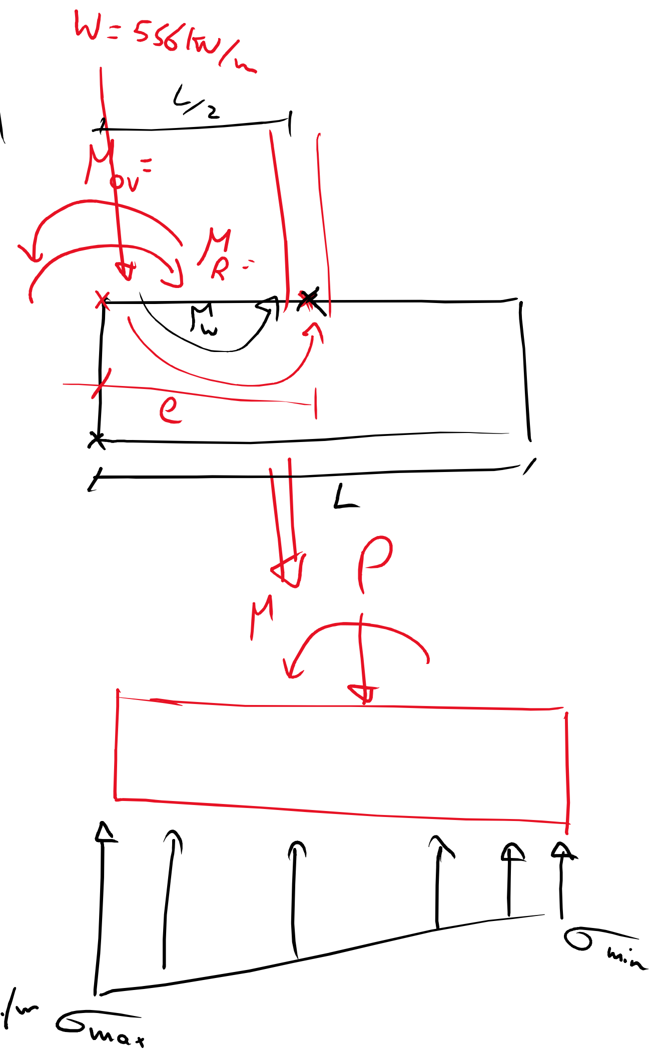
③ Bearing Capacity Check

$$\underline{\underline{\sigma_{serv}}} \leq q_{all} \text{ for Soil}$$



Sum of moment

$$(+)\ M_s = M_{or} - M_R + W \times \frac{L}{2}$$

$$= 433 - 1290 + 556 \times \left(\frac{4.2}{2}\right)$$

$$= 310.6 \ kN.m/m \ (+)$$

$$\sigma_{max} = \frac{P}{A}\left(1 + \frac{6e}{L}\right)$$

$$e = \frac{M_s}{P} = \frac{310.6}{556} = 0.56 m$$

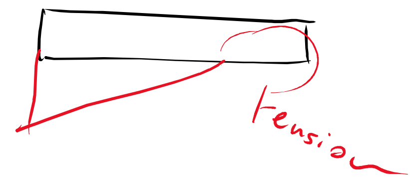
$$\sigma_{max} = \frac{556}{\underset{L}{4.2} \times 1m} \times \left(1 + \frac{6 \times 0.56}{4.2}\right) = 238 \ kN/m \ \sigma_{max} < q_{all} = 250 \ \underline{ok}$$

$\sigma_{min} \geq 0$ to avoid tension.

$$\sigma_{min} = \frac{556}{4.2}\left(1 - \frac{6 \times 0.56}{4.2}\right) = 27 \ kN/m^2 > 0 \quad \text{no tension}$$

Notes

$\sigma_{min} < 0 \Rightarrow$ redesign to avoid tension



$\sigma_{max} > q_{all} \rightarrow$ toe length $\rightarrow$ increase

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Strength failure modes

→ we use factored load for moment and shear

→ applied dead load (soil or concrete) → use 1.2

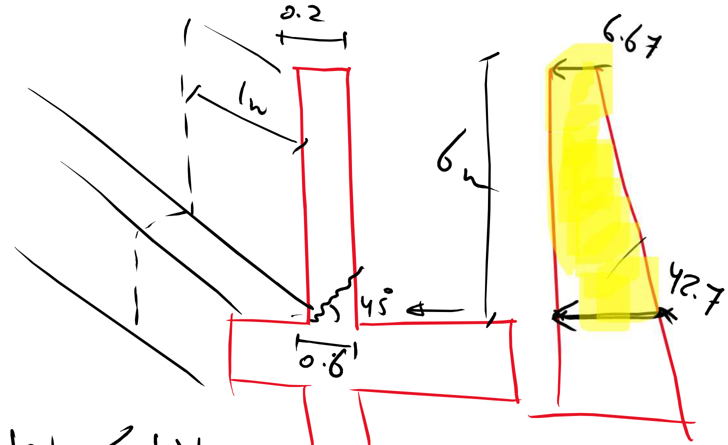
→ resisting dead load → 0.9 or neglect it

→ applied live load (soil pressure, surcharge load) → use 1.6

→ resisting live load → neglect it

Check shear and moment capacity

Check Shear in Stem



$$\phi V_c = 0.75\left(\frac{1}{6}\sqrt{25} \times \frac{(520)\times \underline{\underline{1000}}^{b}}{1000}\right)$$

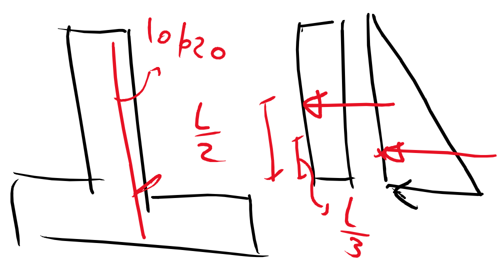
$$= 325 \text{ kN/m}$$

$$\underline{\underline{P_u}} = \left(\frac{6.67+42.7}{2}\right)\times 6 \times \underset{=}{\overset{F.S}{1.6}} = 237 \text{ kN} < \phi V_c \;\; \underline{\underline{ok}}$$

↳ not V_u (V_u at d from face of member)

Steel in Stem

$$M_u = \underset{F.S}{1.6}\left(\frac{6.67\times 6^2}{2}\right) + \frac{36\times 6}{2}\times\frac{6}{3}\Big) = 538 \text{ kN.m/m}$$



$\Rightarrow$ d = 520 mm, b = 1000

$\Rightarrow \rho = 5.56\times10^{-3} \rightarrow A_s = \rho b d$

$= 2890 \text{ mm}^2/\text{m}$

Check $A_{s\,min} = \overset{0.0033}{\frac{1.4}{f_y}} \times bd = \frac{1.4}{420}\times 1000 \times 520 = 1733 \text{ mm}^2 < \underline{\underline{A_s}}$ ok.

$\Rightarrow$ use 10 φ20 mm for Stem back vertical Reinforcement

if wall thickness $\geq$ 250 mm use two layer reinforcement

* $A_{s\,shrinkage}$ (vertical) $= 0.0012bh$

$$= \frac{0.0012\times 1000\times 600}{\underline{\underline{2}}} = 360 \text{ mm}^2/\text{m}$$

for one layer

Use 6φ12 mm/m

increase to avoid Crack.

horizontal reinforcement in stem

use two layers each of has

$$\frac{A_{s\,min}}{2} = \frac{0.002\,bh}{2} = \frac{0.002 \times 1000 \times 600}{2}$$

$600\ mm^2/m \rightarrow$ use $6\phi 12/m$ each layer