

اول فصل

Site Investigation

د ساحی ٹیسٹ کول (تحقیقات)

Site Investigation



د معیاری ساختمان لپاره
بنه ساحه ټاکنه ضروره دی



Ketabton.com

?

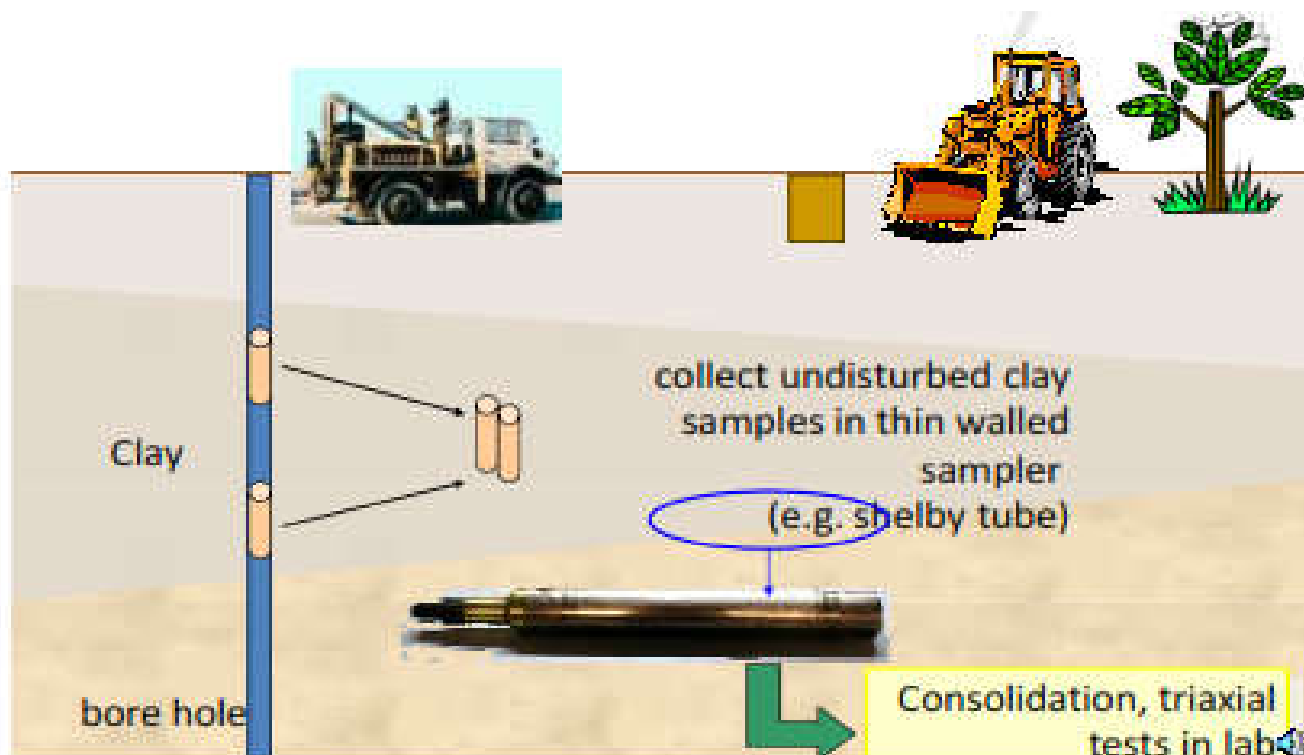
What is the soil problem ??

e.g., reactive clays, soft soils, sink holes, etc.

د خاورو مشکلات څه دی ???

فعالی خاوری ، نرمی خاوری او د ځمکی
لاندي غارونه

د ساحی ٹیسٹول (تحقیقاتو) تکلاری



❖ مقدماتی (لمرنی) کتنه د ساحه نه

❖ د ساحی تحقیقات

❖ د ساحی په بارکی معلومات

❖ د موادو په لابراتوار کی ٹیسٹ کول

❖ د جیوتکنیکل تحلیل او تفسیر

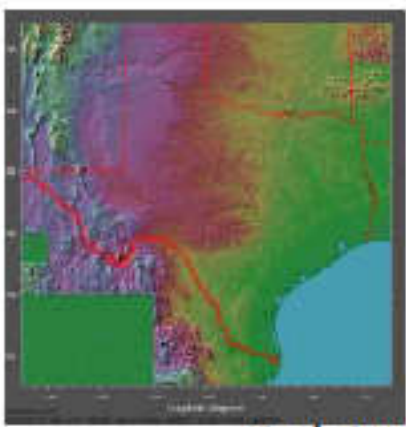
❖ د ساحی معلوماتو رپورت جوړول

❖ مقدماتی (لمړنی) کتنه د ساحه نه



- a. ساحی ته لاسرسیه
- b. توپوگرافی
- c. جیولوجی معلومات
- d. د همسایه ګانو سره توافق
- e. د ترافیک کنترول
- f. د ساحه آبرو
- g. دغه ځمکې کوم بڼه والی لری
- h. د ساحه شیب (همواری)
- i. آوبه

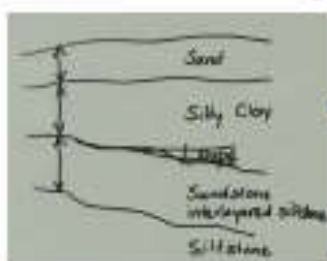
❖ د ساحی تحقیقات



- a. د ساحه توسعه پلان
- b. د ساحه ماسټر پلان
- c. د محلی سایت نقشی
- d. د توپوگرافی نقشی
- e. د ستلاپت عکسونه (نقشه)
- f. د جیولوجیکی نقشه
- g. د ځمکې لاندی د ټیسټونو (تحقیقاتو) رپورت

❖ د ځمکې لاندې تحقیقاتونه مطلب څه شي دی

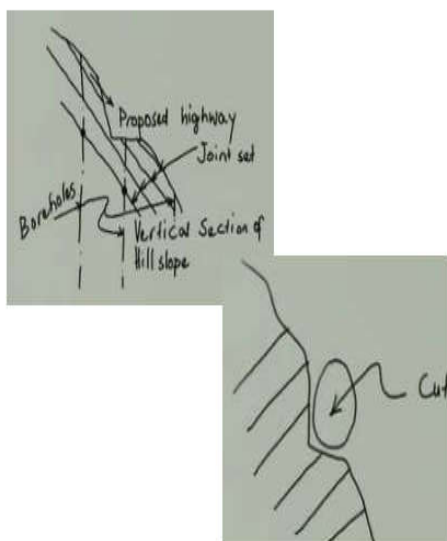
- a. د ځمکې قشرونه معلومول او مشخص کول دی
- b. د هر قشر تشریح او مشخصات معلومول
- c. د تراکم ، د هوا خلاوی، کثافت او نور مقدار معلومول
- d. د ځمکې عوارض پیدا کول لکه ، درزونه ، خلاوی ، ضیف قشرونه او نور
- e. د ساده او عالی آزمایشتو (تیسټونه)
- f. د هیدروژنولوژیکی شرایطو ارزیابی



❖ د بورنو (Bores) تعداد او عمق

تر نن پوری کوم معین تعداد او معین عمق په کوم سټینډرډ که نشته ،

۱- معمولاً تعداد د بورنو په لاندې لاملونو پور تړلی دی



a. د خاوره تغیرات

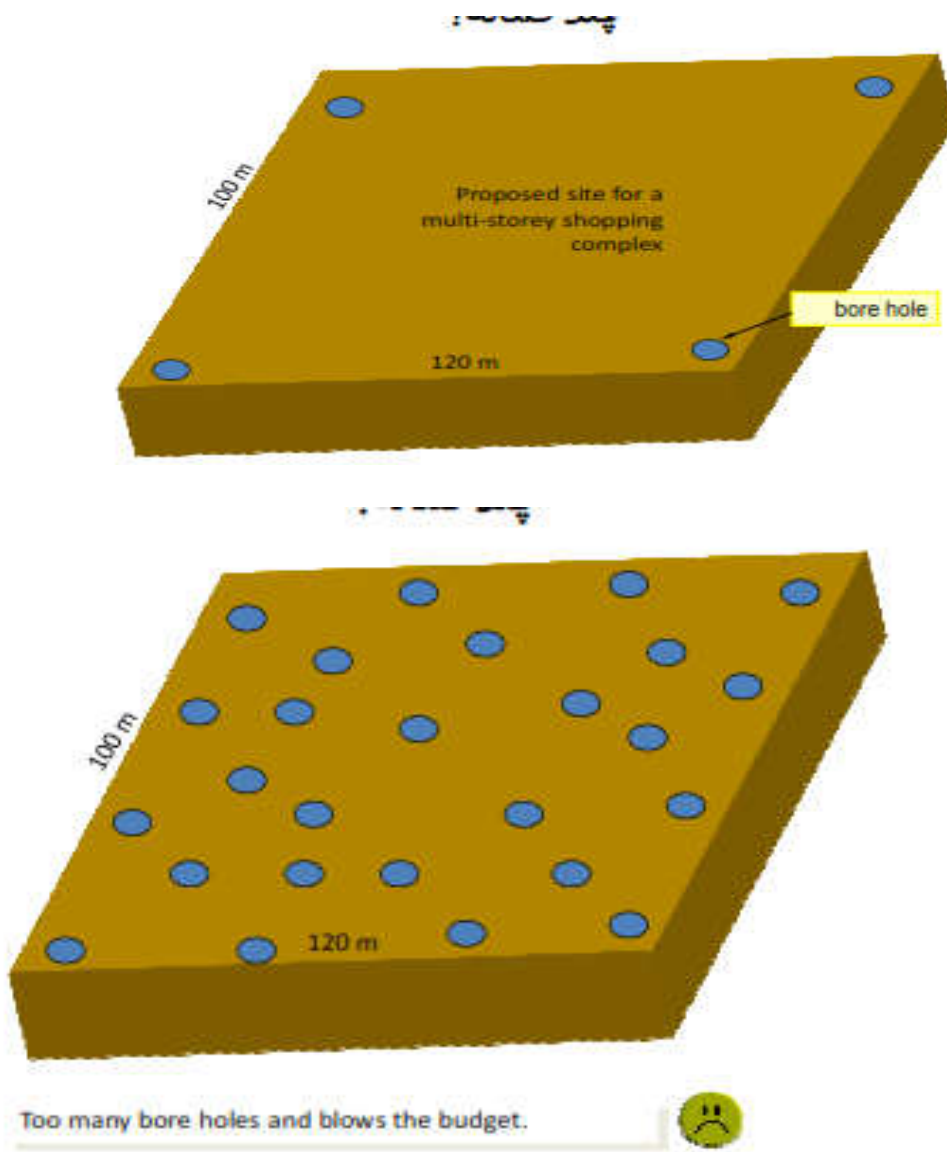
b. د بار زیاتوالی

c. د ساختمان مهم والی او حیاتی

۲- د تیرو تجروبو پر بنیاد

a. نرمی خاورو لپاره ۳۰ نه تر ۶۰ مترو

b. په سختو خاورو که امکان لری تر ۱۵۰ مترو پوری ورسیری



په بعضی کتابونو که د لاندی جدولونو څخه استفادی کیری

Subsurface Conditions	Structure Footprint Area for Each Exploratory Boring	
	(m ²)	(ft ²)
Poor quality and/or erratic	100–300	1,000–3,000
Average	200–400	2,000–4,000
High quality and uniform	300–1,000	3,000–10,000

Subsurface Conditions	Minimum Depth of Borings (S = number of stories; D = anticipated depth of foundation)	
	(m)	(ft)
Poor	$6 S^{0.7} + D$	$20 S^{0.7} + D$
Average	$5 S^{0.7} + D$	$15 S^{0.7} + D$
Good	$3 S^{0.7} + D$	$10 S^{0.7} + D$

Areas of investigation	Boring layout
New site of wide extent	Space preliminary borings 60 to 150 m (200 to 500 ft) apart so that area between any four borings includes approximately 10 percent of total area. In detailed exploration, add borings to establish geological sections at the most useful orientations.
Development of site on soft compressible soil	Space borings 30 to 60 m (100 to 200 ft) at possible building locations. Add intermediate borings when building site is determined.
Large structure with separate closely spaced footings	Space borings approximately 15 m (50 ft) in both directions, including borings at possible exterior foundation walls, at machinery or elevator pits, and to establish geologic sections at the most useful orientations.

Low-load warehouse building of large area	Minimum of four borings at corners plus intermediate borings at interior foundations sufficient to define subsoil profile.
Isolated rigid foundation	For foundation 230 to 930 m ² (2500 to 10,000 ft ²) in area, minimum of three borings around perimeter. Add interior borings, depending on initial results.
Isolated rigid foundation	For foundation less than 230 m ² (2500 ft ²) in area, minimum of two borings at opposite corners. Add more for erratic conditions.
Major waterfront structures, such as dry docks	If definite site is established, space borings generally not farther than 15 m (50 ft), adding intermediate borings at critical locations, such as deep pump well, gate seat, tunnel, or culverts.
Long bulkhead or wharf wall	Preliminary borings on line of wall at 60-m (200-ft) spacing. Add intermediate borings to decrease spacing to 15 m (50 ft). Place certain intermediate borings inboard and outboard of wall line to determine materials in scour zone at toe and in active wedge behind wall.
Cut stability, deep cuts, and high embankments	Provide three to five borings on line in the critical direction to provide geological section for analysis. Number of geologic sections depends on extent of stability problem. For an active slide, place at least one boring upslope of sliding area.
Dams and water-retention structures	Space preliminary borings approximately 60 m (200 ft) over foundation area. Decrease spacing on centerline to 30 m (100 ft) by intermediate borings. Include borings at location of cutoff, critical spots in abutment, spillway, and outlet works.
Source: From NAVFAC DM-7.1, 1982.	

Areas of Investigation	Recommended Boring Depth
Bridge Foundations* Highway Bridges	
1. Spread Footings	<p>For isolated footings of breadth L_f and width $\leq 2B_f$, where $L_f \leq 2B_f$, borings shall extend a minimum of two footing widths below the bearing level.</p> <p>For isolated footings where $L_f \geq 5B_f$, borings shall extend a minimum of four footing widths below the bearing level.</p> <p>For $2B_f \leq L_f \leq 5B_f$, minimum boring length shall be determined by linear interpolation between depths of $2B_f$ and $5B_f$ below the bearing level.</p>
2. Deep Foundations	<p>In soil, borings shall extend below the anticipated pile or shaft tip elevation a minimum of 6 m, or a minimum of two times the maximum pile group dimension, whichever is deeper.</p> <p>For piles bearing on rock, a minimum of 3 m of rock core shall be obtained at each boring location to verify that the boring has not terminated on a boulder.</p> <p>For shafts supported on or extending into rock, a minimum of 3 m of rock core, or a length of rock core equal to at least three times the shaft diameter for isolated shafts or two times the maximum shaft group dimension, whichever is greater, shall be extended below the anticipated shaft tip elevation to determine the physical characteristics of rock within the zone of foundation influence.</p>
Retaining Walls	<p>Extend borings to depth below final ground line between 0.75 and 1.5 times the height of the wall. Where stratification indicates possible deep stability or settlement problem, borings should extend to hard stratum.</p> <p>For deep foundations use criteria presented above for bridge foundations.</p>
Roadways	Extend borings a minimum of 2 m below the proposed subgrade level.
Cuts	Borings should extend a minimum of 5 m below the anticipated depth of the cut at the ditch line. Borings depths should be increased in locations where base stability is a concern due to the presence of soft soils, or in locations where the base of the cut is below groundwater level to determine the depth of the underlying pervious strata.
Embankments	Extend borings a minimum depth equal to twice the embankment height unless a hard stratum is encountered above this depth. Where soft strata are encountered which may present stability or settlement concerns the borings should extend to hard material.
Culverts	Use criteria presented above for embankments.
*Note: Taken from AASHTO Standard Specifications for Design of Highway Bridges	

<i>Geotechnical Features</i>	<i>Boring Layout</i>
Bridge Foundations	<p>For piers or abutments over 30 m wide, provide a minimum of two borings.</p> <p>For piers or abutments less than 30 m wide, provide a minimum of one boring.</p> <p>Additional borings should be provided in areas of erratic subsurface conditions.</p>
Retaining Walls	<p>A minimum of one boring should be performed for each retaining wall. For retaining walls more than 30 m in length, the spacing between borings should be no greater than 60 m. Additional borings inboard and outboard of the wall line to define conditions at the toe of the wall and in the zone behind the wall to estimate lateral loads and anchorage capacities should be considered.</p>
Roadways	<p>The spacing of borings along the roadway alignment generally should not exceed 60 m. The spacing and location of the borings should be selected considering the geologic complexity and soil/rock strata continuity within the project area, with the objective of defining the vertical and horizontal boundaries of distinct soil and rock units within the project limits.</p>
Cuts	<p>A minimum of one boring should be performed for each cut slope. For cuts more than 60 m in length, the spacing between borings along the length of the cut should generally be between 60 and 120 m.</p> <p>At critical locations and high cuts, provide a minimum of three borings in the transverse direction to define the existing geological conditions for stability analyses. For an active slide, place at least one boring upslope of the sliding area.</p>
Embankments	<p>Use criteria presented above for Cuts.</p>
Culverts	<p>A minimum of one boring at each major culvert. Additional borings should be provided for long culverts or in areas of erratic subsurface conditions.</p>

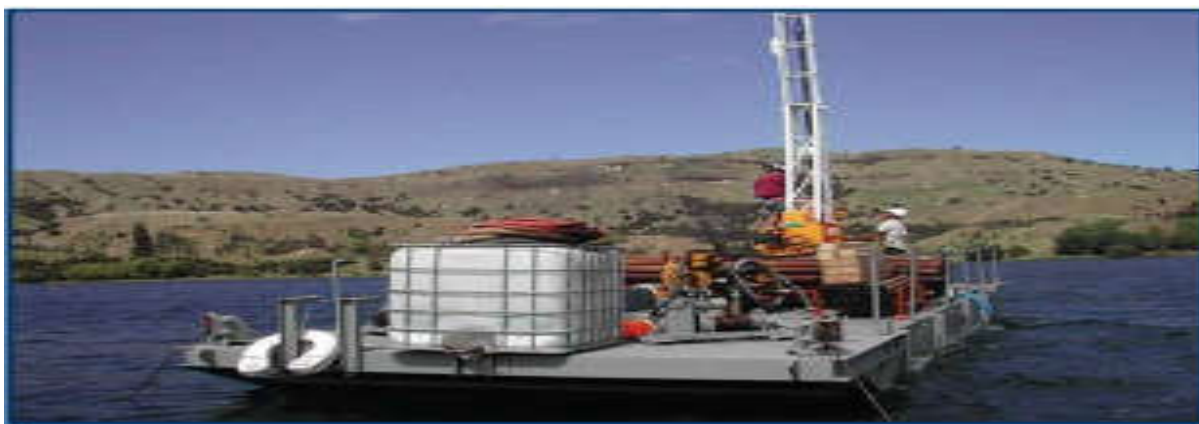
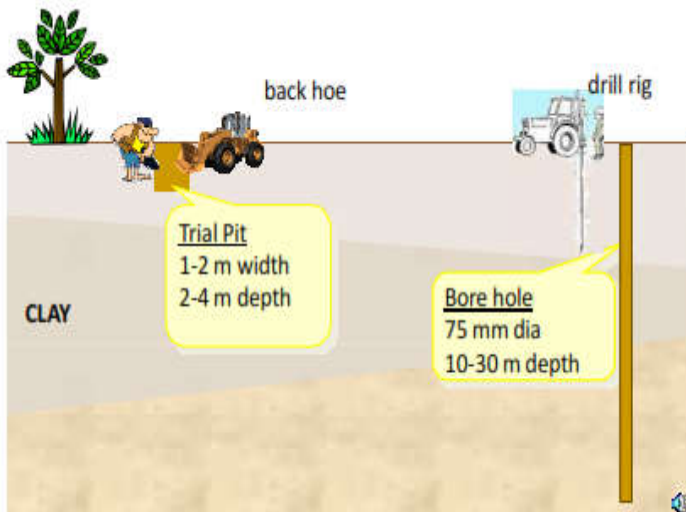
*Also see *FHWA Geotechnical Checklist and Guidelines*; FHWA-ED-88-053

Geotechnical Feature	Minimum Number of Borings	Minimum Depth of Borings
Structure foundation	One per substructure unit for width ≤ 30 m Two per substructure unit for width > 30 m	Advance borings: (1) through unsuitable foundation soils (e.g., peat, highly organic soils, soft fine-grained soils) into competent material of suitable bearing capacity; (2) to a depth where stress increases due to estimated footing load is less than 10% of the existing effective soil overburden stress; or (3) a minimum of 3 m into bedrock if bedrock is encountered at shallower depth
Retaining walls	Borings alternatively spaced every 30 to 60 m in front of and behind wall	Extend borings to depth of two times wall height or a minimum of 3 m into bedrock
Culverts	Two borings depending on length	See structure foundations
Bridge approach embankments over soft ground	For approach embankments placed over soft ground, one boring at each embankment to determine problems associated with stability and settlement of the embankment (note: borings for approach embankments are usually located at proposed abutment locations to serve a dual function)	See structure foundations Additional shallow explorations at approach embankment locations are an economical means to determine depth of unsuitable surface soils
Cuts and embankments	Borings typically spaced every 60 (erratic conditions) to 150 m (uniform conditions) with at least one boring taken in each separate landform. For high cuts and fills, two borings along a straight line perpendicular to centerline or planned slope face to establish geologic cross section for analysis	Cut: (1) in stable materials, extend borings a minimum of 3 to 5 m below cut grade (2) in weak soils, extend borings below cut grade to firm materials, or to the depth of cut below grade whichever occurs first Embankment: extend borings to firm material or to depth of twice the embankment height

Source: Modified after FHWA, 1993, Soils and Foundations, Workshop Manual, 2nd ed., FHWA HI-88-009, National Highway Institute, NHI Course No. 13212, Revised, July.

د نمونه راخستلو لاری

- a. لاس خورلی نمونه
- b. لاس نه خورلی نمونه



اس پی ٹی

Standard Penetration Test (SPT)

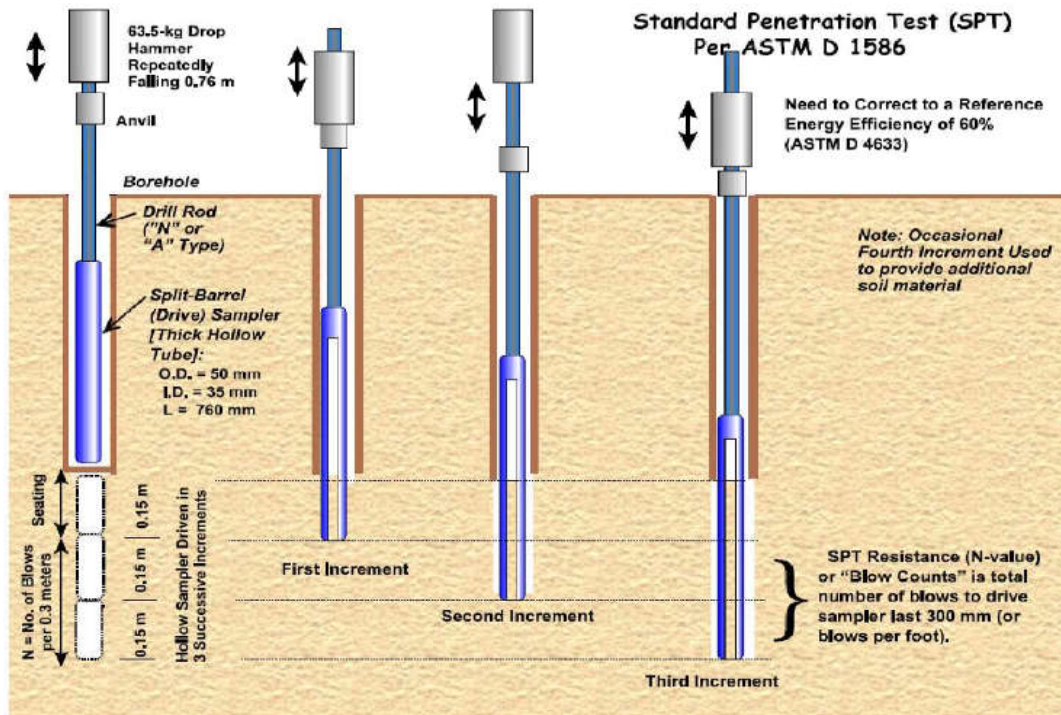


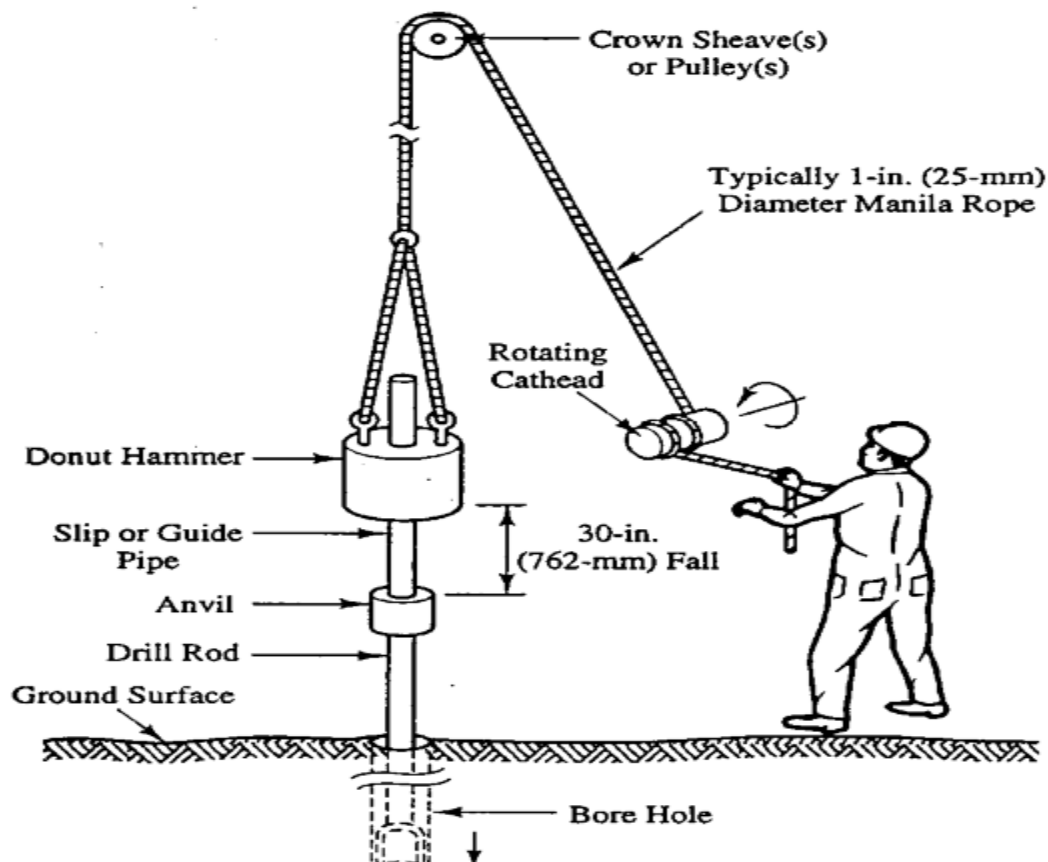
- a. پھ دی ازمایشت که یو اله چه د غوچیونکی میله په سر وصل دی په خُمکې که داخلیری اود هر قشر مقاومت او اندازگیری کوی
- b. دغه مقاومت نظر د جیوتکنیکل پارمٹرو پوری تړل دی لکه د اصطکاک زاویه ، د خاوری سختی او نور
- c. د ازمایشت ۸۰٪ د تعداد د دیزاین لپاره پکار کیری

SPT وسایل

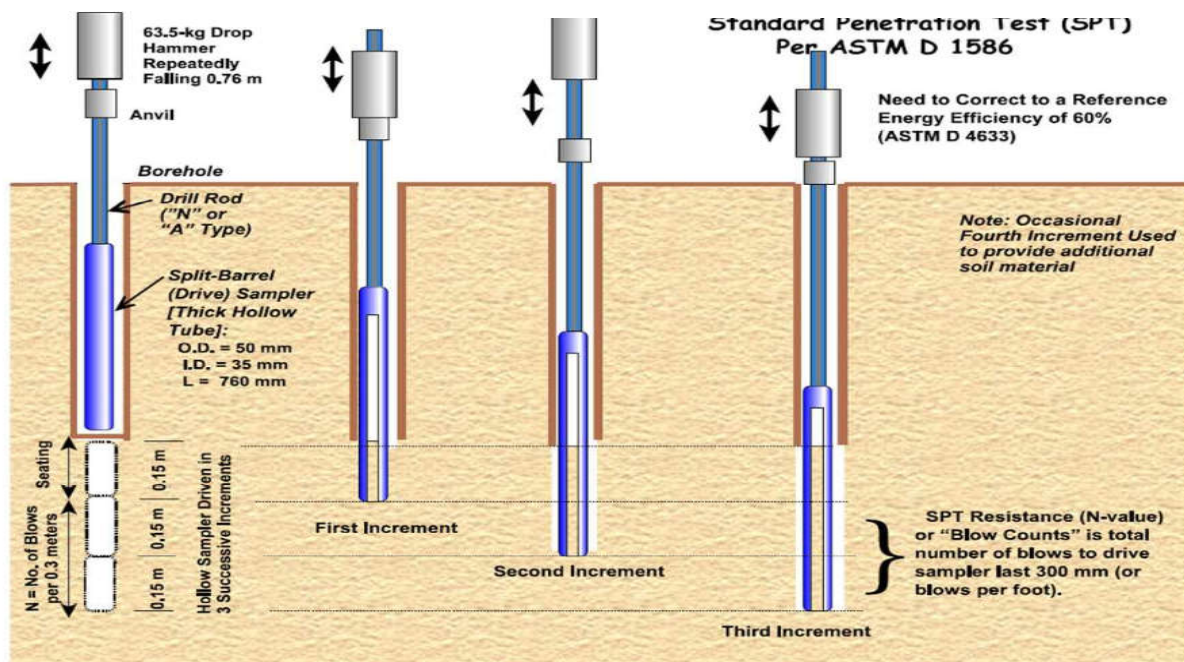
- a. چکش ۶۳,۵ کیلوگرام
- b. د چکش سقوط ارتفاع ۷۶ سانتی متر
- c. د نمونہ راخستلو اندازه ۴۵ سانتی متر
- d. د صرباتو تعداد د هر ۱۵ سانتی متر نفوذ (داخلیدلو) لپاره یادداشت شی

SPT Hammer





- Obtain
- Simple
- Suitable
- Can be
- Available



Standard Split Spoon Sampler



د SPT څخه کوم موارد (ارقام) لاسته راوړه شوی؟

a. D_r (نسبتی تراکم) لاسته راوړو

b. تعیین ϕ

c. C تعیین

d. خاورو نوعیت او خواص

e. د برشی امواجو سرعت

Soil Type	D_r	Ψ	K_0	OCR	S_t	s_u	ϕ	E,G	M	G_0	k	c_H
Sand	3-4	4		5			3-4	4-5		4-5		
Clay		5	5	4	5	3-4	5	4-5	5	4-5	5	5

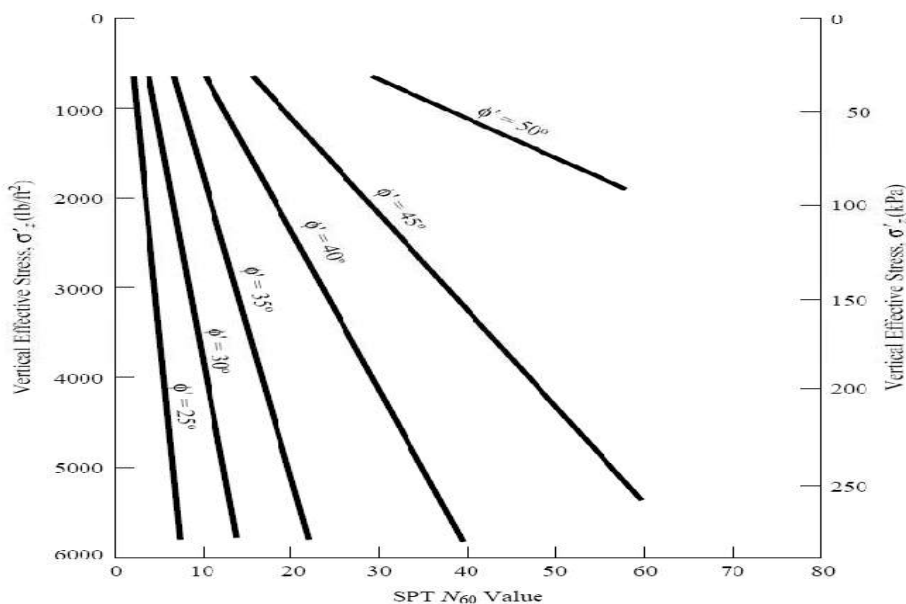
1 = high; 2 = high to moderate; 3 = moderate; 4 = moderate to low; 5 = low;
Blank = no applicability

Where:

D_r	Relative density	ϕ	Friction angle
Ψ	State Parameter	K_0	In-situ stress ratio
E, G	Young's and Shear moduli	G_0	Small strain shear moduli
OCR	Over consolidation ratio	M (or m_v)	Compressibility
s_u	Undrained shear strength	S_t	Sensitivity
c_H	Coefficient of consolidation	k	Permeability

Type of Soil	Pile Design	Bearing Capacity	Settlement	Compaction Control	Liquefaction
Sand	2-3	1-2	2-3	2-3	1-2
Clay	3-4	3-4	4-5	4-5	1-2
Intermediate Soils	3-4	2-3	3-4	4-5	1-2

Reliability rating: 1 = High; 2 = High to moderate; 3 = Moderate; 4 = Moderate to low; 5 = low



Schmertman(1975) :

$$\phi' = \tan^{-1} \left[\frac{N_{60}}{12.2 + 20.3 \left(\frac{\sigma'_0}{P_a} \right)} \right]^{0.34}$$

σ'_0 = effective overburden stress

P_a = atmospheric pressure in the same unit as σ'_0

($\approx 100 \text{ kN/m}^2$ or 2000 lb/ft^2)

Peck, Hanson and Thornburn (1974):

$$\phi'(\text{deg}) = 27.1 + 0.3(N_{60})_{\text{cor}} - 0.00054[(N_{60})_{\text{cor}}]^2$$

Hatanaka and Uchida (1996):

$$\phi' = \sqrt{20(N_{60})_{\text{cor}} + 20}$$

Stroud (1974): $s_u = KN_{60}$

s_u = Undrained shear strength of clay

K=Constant=3.5-6.5 kN/m² (0.507 – 0.942 lb/in²) average 4.4 kN/m²

Hara et al. (1971): $c_u \text{ (kN/m}^2\text{)} = 29N_{60}^{0.5}$

Mayne and Kemper (1988): $OCR = 0.193 \left(\frac{N_{60}}{\sigma'_v} \right)^{0.5}$

σ'_v = effective vertical stress in MN/m²

TABLE 7.4 Correlation of N , N_{60} , γ , D_r , and ϕ' for Coarse-Grained Soils

N	N_{60}	Compactness	γ (kN/m ³)	D_r (%)	ϕ' (degrees)
0-4	0-3	Very loose	11-13	0-20	26-28
4-10	3-9	Loose	14-16	20-40	29-34
10-30	9-25	Medium	17-19	40-70	35-40*
30-50	25-45	Dense	20-21	70-85	38-45*
>50	>45	Very dense	>21	>85	>45*

*These values correspond to ϕ'_{cr}

TABLE 7.5 Correlation of N_{60} and s_u for Saturated Fine-Grained Soils

N_{60}	Description	s_u (kPa)
0-2	Very soft	<10
3-5	Soft	10-25
6-9	Medium	25-50
10-15	Stiff	50-100
15-30	Very stiff	100-200
>30	Extremely stiff	>200

SPT Correlations in Granular Soils

$(N)_{60}$	D_r (%)	consistency
0-4	0-15	very loose
4-10	15-35	loose
10-30	35-65	medium
30-50	65-85	dense
>50	85-100	very dense

not corrected for overburden

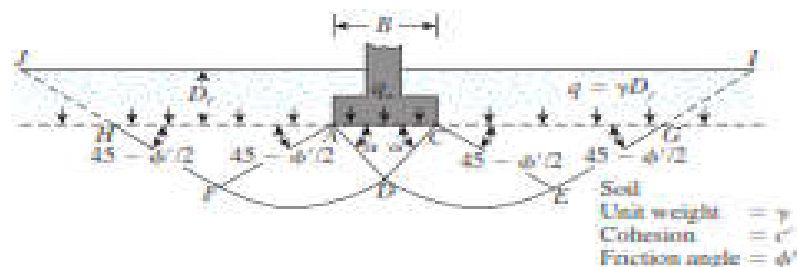


Table 3.1 Terzaghi's Bearing Capacity Factors—Eqs. (3.4), (3.5), and (3.6) a From Kumbhojkar (1993)

ϕ'	N_c	N_q	N_{γ}^a	ϕ'	N_c	N_q	N_{γ}^a
0	5.70	1.00	0.00	26	27.09	14.21	9.84
1	6.00	1.10	0.01	27	29.24	15.90	11.60
2	6.30	1.22	0.04	28	31.61	17.81	13.70
3	6.62	1.35	0.06	29	34.24	19.98	16.18
4	6.97	1.49	0.10	30	37.16	22.46	19.13
5	7.34	1.64	0.14	31	40.41	25.28	22.65
6	7.73	1.81	0.20	32	44.04	28.52	26.87
7	8.15	2.00	0.27	33	48.09	32.23	31.94
8	8.60	2.21	0.35	34	52.64	36.50	38.04
9	9.09	2.44	0.44	35	57.75	41.44	45.41
10	9.61	2.69	0.56	36	63.53	47.16	54.36
11	10.16	2.98	0.69	37	70.01	53.80	65.27
12	10.76	3.29	0.85	38	77.50	61.55	78.61
13	11.41	3.63	1.04	39	85.97	70.61	95.03
14	12.11	4.02	1.26	40	95.66	81.27	115.31
15	12.86	4.45	1.52	41	106.81	93.85	140.51
16	13.68	4.92	1.82	42	119.67	108.75	171.99
17	14.60	5.45	2.18	43	134.58	126.50	211.56
18	15.12	6.04	2.59	44	151.95	147.74	261.60
19	16.56	6.70	3.07	45	172.28	173.28	325.34
20	17.69	7.44	3.64	46	196.22	204.19	407.11
21	18.92	8.26	4.31	47	224.55	241.80	512.84
22	20.27	9.19	5.09	48	258.28	287.85	650.67
23	21.75	10.23	6.00	49	298.71	344.63	831.99
24	23.36	11.40	7.08	50	347.50	415.14	1072.80
25	25.13	12.72	8.34				

*From Kumbhojkar (1993)

Table 3.2 Terzaghi's Modified Bearing Capacity Factors N_c , N_q , and N_{γ}

ϕ'	N_c	N_q	N_{γ}	ϕ'	N_c	N_q	N_{γ}
0	5.70	1.00	0.00	26	15.53	6.05	2.59
1	5.90	1.07	0.005	27	16.30	6.54	2.88
2	6.10	1.14	0.02	28	17.13	7.07	3.29
3	6.30	1.22	0.04	29	18.03	7.66	3.76
4	6.51	1.30	0.055	30	18.99	8.31	4.39
5	6.74	1.39	0.074	31	20.03	9.03	4.83
6	6.97	1.49	0.10	32	21.16	9.82	5.51
7	7.22	1.59	0.128	33	22.39	10.69	6.32
8	7.47	1.70	0.16	34	23.72	11.67	7.22
9	7.74	1.82	0.20	35	25.18	12.75	8.35
10	8.02	1.94	0.24	36	26.77	13.97	9.41
11	8.32	2.08	0.30	37	28.51	15.32	10.90
12	8.63	2.22	0.35	38	30.43	16.85	12.75
13	8.96	2.38	0.42	39	32.53	18.56	14.71
14	9.31	2.55	0.48	40	34.87	20.50	17.22
15	9.67	2.73	0.57	41	37.45	22.70	19.75
16	10.06	2.92	0.67	42	40.33	25.21	22.50
17	10.47	3.13	0.76	43	43.54	28.06	26.25
18	10.90	3.36	0.88	44	47.13	31.34	30.40
19	11.36	3.61	1.03	45	51.17	35.11	36.00
20	11.85	3.88	1.12	46	55.73	39.48	41.70
21	12.37	4.17	1.35	47	60.91	44.45	49.30
22	12.92	4.48	1.55	48	66.80	50.46	59.25
23	13.51	4.82	1.74	49	73.55	57.41	71.45
24	14.14	5.20	1.97	50	81.31	65.60	85.75
25	14.80	5.60	2.25				

که د دغه ساحه تحقیقات په وخت شوی وایی!!!!



نو نن به د پیزا مانی داسی نه وایی

دوهم فصل

تهداب (Foundation)

تهداب د ساختمان هغه برخه ده چې پر ساختمان وارد شوی بار د خاورې هغه برخې ته لیردوي چې تهداب پرې پروت دی ، په دې کې د ځمکې د لیول له سطحې څخه لاندې برخې (Sub structure) او هغه اضافي برخې چې د ساختمان په شمول وزنونه لاندې خاورې ته لیردوي شاملې دي ،

او Sub Structure د تهاداب هغه برخه ده چې د ځمکې د لیول له سطحې د ساختمان د پورته برخې (Super structure) لپاره بنیاد یا قاعدې دنده تر سره کوي.

زیات د استفاده وړ اصطلاحات د تهاداب ډیزاین کې

1:- تهاداب (Foundation) - تهاداب د ساختمان هغه برخه ده چې پر ساختمان وارد شوی بار د خاورې هغه برخې ته لیردوي او د اساس په نوم هم یادېږي.

2:- سپل (Footing) - سپل عبارت د تهاداب له یو برخې څخه دی چې د ځمکې سر مستقیم په تماس کې دی

3:- د تهاداب خاوره (Foundation Soil) - عبارت د ځمکې خارجي سطح څخه دی چې په تهاداب وارد شوی بار احجار یا لاندی سطح ته انتقالوي.

4:- د خاورې برداشت قابلیت (د خاوره د زغم قوی) (Soil Bearing Capacity) - عبارت له هغه زغمونکې توان (توان تحمل پذیري) څخه دی چې خاورې یا احجار یې د قوی په مقابل کې لری

ترتیب کوونکی:- انجینر عبدالحمید (رسولی)
مضمون: د تھداب دیزاین

6:- مجموعی شدت د فشار (Gross Pressure intensity)- عبارت له مجموعی فشار په تھداب

یا سپل باندی په شمول د ټولو وزنونو لکه د سپل وزن ، د پرکاری خاوری وزن او داسی نور

7:- خالص شدت د فشار (Net Pressure intensity)- عبارت له مجموعی شدت د فشار بیدون

دی اضافی خاورونه لکه

$$q_n = q - \gamma * D_f$$

q -مجموعی شدت د فشار

γ -د خاوری حجم ورن

D_f - د تھدات عمق (ژوروالی)

8:- نهایی د خاوری برداشت قابلیت (Ultimate Bearing Capacity)-نهایی برداشت قابلیت

عبارت له هغه اصغری مجموعی فشار په تھداب باندی دی چی خاوری برش ونکړی دی ، او
نشست هم د مجازی حد څخه زیات نشی .

9:-مصوون خالص د خاوری برداشت قابلیت (Net Safe Bearing Capacity)- عبارت د نهایی

د خاوری برداشت قابلیت تقسیم پر یو د مصوونیت فکتور (Factor of Safety) باندی لکه :

$$q_{ns} = q_{ult} / SF$$

10:- مجازی فشار قابل د تحمل (زغم) (Allowable Bearing Pressure)-عبارت له خالص بار

شدت چه نه د خاوری برش باعث شی او نه د مجازی نشست له حد زیات شی .

Bearing Capacity Analyses for foundation

تحليل برداشت قابليت د تهاداب لپاره

ویسیک (Vesic 1940-1973) د پنځلس نه زیاتی معادلی (تیوری) دخاوری برداشت قابليت ته ترتیب کړی مکر یو معادلی (تیوری) هم نن سبا قابل د استفادی نه دی ، په همدی وختونو که (1943) پراندیل (Prandtl) په یو معادله (تیوری) کار شروع کړی مکر تکمیل وایی ونشو ، ترزاغی (Terzaghi) د پراندیل معادلی ته انکشاف ورکړ او یو معادلی د خاوری برداشت قابليت لپاره ترتیب کړل چه تر نن پور قابل د استفادی ده ، او دغه معادله (تیور) د ترزاغی معادله په نوم یادیری ، او د کم عمق تهادابونو لپاره کارول کیری ، یعنی $H \leq B$ ، H-د تهاداب ارتفاع او B- د تهاداب عرض دی ، د ترزاغی معادله (تیوری) ته نور پوهانو لکه مایراوف ، هانسن او وزیک انکشاف او یا همدا شان معادله ترتیب کړی خو نن سبا د مایراوف معادله کوم چي د ترزاغی معادله ته یی د عمق ، شکل او د قوی زاویه په نظر که نیول دی ډیر زیات د استفادی وړ دی ، او دغه معادله د عمومی (ترزاغی-مایراوف) معادلی په نوم یادیری.

نوټ: لاندنی جدول صرف د ترزاغی معادلو لپاره استعمالیری او د یادونی وړ دی چه د اصطحاکاک زاویہ () او د چسپیش ضریب (C) په ساختمانی لابراتوارنو کی د یو خاص ماشین الاتو په واسطه پیدای کیری او د جیوتکنیکل رپورت په واسطه د انجینرانو ته ورکول کیری.

3.3 Terzaghi's Bearing Capacity Theory 139

Table 3.1 Terzaghi's Bearing Capacity Factors—Eqs. (3.4), (3.5), and (3.6) a From Kumbhojkar (1993)

ϕ'	N_c	N_q	N_γ^a	ϕ'	N_c	N_q	N_γ^a
0	5.70	1.00	0.00	26	27.09	14.21	9.84
1	6.00	1.10	0.01	27	29.24	15.90	11.60
2	6.30	1.22	0.04	28	31.61	17.81	13.70
3	6.62	1.35	0.06	29	34.24	19.98	16.18
4	6.97	1.49	0.10	30	37.16	22.46	19.13
5	7.34	1.64	0.14	31	40.41	25.28	22.65
6	7.73	1.81	0.20	32	44.04	28.52	26.87
7	8.15	2.00	0.27	33	48.09	32.23	31.94
8	8.60	2.21	0.35	34	52.64	36.50	38.04
9	9.09	2.44	0.44	35	57.75	41.44	45.41
10	9.61	2.69	0.56	36	63.53	47.16	54.36
11	10.16	2.98	0.69	37	70.01	53.80	65.27
12	10.76	3.29	0.85	38	77.50	61.55	78.61
13	11.41	3.63	1.04	39	85.97	70.61	95.03
14	12.11	4.02	1.26	40	95.66	81.27	115.31
15	12.86	4.45	1.52	41	106.81	93.85	140.51
16	13.68	4.92	1.82	42	119.67	108.75	171.99
17	14.60	5.45	2.18	43	134.58	126.50	211.56
18	15.12	6.04	2.59	44	151.95	147.74	261.60
19	16.56	6.70	3.07	45	172.28	173.28	325.34
20	17.69	7.44	3.64	46	196.22	204.19	407.11
21	18.92	8.26	4.31	47	224.55	241.80	512.84
22	20.27	9.19	5.09	48	258.28	287.85	650.67
23	21.75	10.23	6.00	49	298.71	344.63	831.99
24	23.36	11.40	7.08	50	347.50	415.14	1072.80
25	25.13	12.72	8.34				

^aFrom Kumbhojkar (1993)

مثال : - يو مربع شكل تهاداب چې $2m \times 2m$ ابعاد لري په پلان كې لرو او د ساحه جيوټكنيكل رپورټ له مخ لروى

د اصطحكاك زاويه $\phi = 25^\circ$ د خاورى فى واحد حجم وزن $\gamma = 16.5 \frac{KN}{m^3}$

د خاورى چسپش ضريب $C = 20 \frac{KN}{m^2}$ د مصوونيت ضريب $SF=3$

د تهاداب عمق $D_f=1.5m$

په تهاداب باندې دخاورى مجازى فشار (q_{all}) او په تهاداب باندې د خاورى لخوا مجموعى قوى (Q) پيدا كړى .

حل: $q_u = 1.3c'N_c + qN_q + 0.4\gamma BN_\gamma$

كله چې د اصطحكاك زاويه $\phi = 25^\circ$ وي د ترزاغى له جدول لروى

Table 3.1 Terzaghi's Bearing Capacity Factors—Eqs. (3.4), (3.5), and (3.6) a From Kumbhojkar (1993)

ϕ'	N_c	N_q	N_γ^a	ϕ'	N_c	N_q	N_γ^a
20	17.69	7.44	3.64	46	196.22	204.19	407.11
21	18.92	8.26	4.31	47	224.55	241.80	512.84
22	20.27	9.19	5.09	48	258.28	287.85	650.67
23	21.75	10.23	6.00	49	298.71	344.63	831.99
24	23.36	11.40	7.08	50	347.50	415.14	1072.80
25	25.13	12.72	8.34				

$$N_c=25.13$$

$$C' = 20 \frac{KN}{m^2}$$

$$\gamma = 16.5 \frac{kg}{m^3}$$

$$N_q=12.72$$

$$SF=3$$

$$q_u = 1.3c'N_c + qN_q + 0.4\gamma BN_\gamma$$

$$N_\gamma=8.34$$

$$D_f=1.5m$$

$$q_u = (1.3)(20)(25.13) + (1.5 \times 16.5)(12.72) + (0.4)(16.5)(2)(8.34)$$

$$= 653.38 + 314.82 + 110.09 = 1078.29 \text{ kN/m}^2$$

د مجازی فشار د پیدا کولو لپاره لرو

$$q_{all} = \frac{q_u}{SF} = \frac{1078.29}{3} \approx 359.5 \frac{KN}{m^2}$$

د مجموعی قوی لپاره لرو

$$Q_{all} = q_{all} \times B^2 = 359.5 \times 2^2 \approx 1438 KN$$

2:- د مایراوف - ترزاغی معادله (عمومی معادله) د خاوری برداشت قابلیت لپاره

د ترزاغی معادلی ته مایراوف انکشاف ورکړی ، مایراوف د تھداب د شکل ، عمق ضرایب په معادله کی اضاف کړل . چې تیوری او معادلی د ترزاغی په شان دی نو نوی معادله به په لاندی ډول وی

$$q_u = c' N_c F_{cs} F_{cd} F_{ci} + q N_q F_{qs} F_{qd} F_{qi} + \frac{1}{2} \gamma B N_\gamma F_{\gamma s} F_{\gamma d} F_{\gamma i} \quad (3.19)$$

In this equation:

c' = cohesion

q = effective stress at the level of the bottom of the foundation

γ = unit weight of soil

B = width of foundation (= diameter for a circular foundation)

$F_{cs}, F_{qs}, F_{\gamma s}$ = shape factors تھداب د شکل ضریب

$F_{cd}, F_{qd}, F_{\gamma d}$ = depth factors تھداب د عمق ضریب

$F_{ci}, F_{qi}, F_{\gamma i}$ = load inclination factors

N_c, N_q, N_γ = bearing capacity factors

د قوی د عمل زاویه ضریب

د برداشت قابلیت ضریب

د پورتنی معادلی د تهاداب شکل ، د تهاداب عمق او د قوی د عمل زاویه ضرایب د لاندنی روابطو په اساس سر لاسته راخی.

Shape

$$F_{cs} = 1 + \left(\frac{B}{L}\right)\left(\frac{N_q}{N_c}\right)$$

1: د شکل ضرایب

$$F_{qs} = 1 + \left(\frac{B}{L}\right) \tan \phi'$$

$$F_{\gamma s} = 1 - 0.4 \left(\frac{B}{L}\right)$$

2:- د عمق ضرایب څلور حالت لری چې په لاندی ډول دی

For $\phi = 0$:

$$\frac{D_f}{B} \leq 1$$

$$F_{cd} = 1 + 0.4 \left(\frac{D_f}{B}\right)$$

$$F_{qd} = 1$$

$$F_{\gamma d} = 1$$

اول حالت : د تهاداب عمق (D_f) پر د تهاداب عرض (B) کوچنی یا مساوی د 1 سره وی او د اصطحاک زاویه مساوی د صفر سره وی

For $\phi' > 0$:

$$\frac{D_f}{B} \leq 1$$

$$F_{cd} = F_{qd} - \frac{1 - F_{qd}}{N_c \tan \phi'}$$

$$F_{qd} = 1 + 2 \tan \phi' (1 - \sin \phi')^2 \left(\frac{D_f}{B}\right)$$

$$F_{\gamma d} = 1$$

دوهم حالت : د تهاداب عمق (D_f) پر د تهاداب عرض (B) کوچنی یا مساوی د 1 سره وی او د اصطحاک زاویه له صفر څخه زیات وی

$$\text{For } \phi = 0: \quad \frac{D_f}{B} > 1$$

$$F_{cd} = 1 + 0.4 \underbrace{\tan^{-1}\left(\frac{D_f}{B}\right)}_{\text{radians}}$$

$$F_{qd} = 1$$

$$F_{\gamma d} = 1$$

دریم حالت: د تھداب عمق (D_f) پر د تھداب عرض (B) لوی د 1 وی او د اصطحکاک زاویه له صفر سره مساوی وی

$$\text{For } \phi' > 0: \quad \frac{D_f}{B} > 1$$

$$F_{cd} = F_{qd} - \frac{1 - F_{qd}}{N_c \tan \phi'}$$

$$F_{qd} = 1 + 2 \tan \phi' (1 - \sin \phi')^2 \underbrace{\tan^{-1}\left(\frac{D_f}{B}\right)}_{\text{radians}}$$

$$F_{\gamma d} = 1$$

دریم حالت: د تھداب عمق (D_f) پر د تھداب عرض (B) لوی د 1 وی او د اصطحکاک زاویه له صفر سره مساوی وی

3:- قوی په زاویه ضرایب

نوټ: که قوی په تھداب باندي عمود وی نو ضرایب به مساوی د 1 سره وی

Inclination

$$F_{ci} = F_{qi} = \left(1 - \frac{\beta^\circ}{90^\circ}\right)^2$$

$$F_{\gamma i} = \left(1 - \frac{\beta}{\phi'}\right)$$

β = inclination of the load on the foundation with respect to the vertical

مثال : - یو مربع شکل تھداب چې $2m \times 2m$ ابعاد لری په پلان کې لرو او د ساحه جیوتکنیکل رپورټ له مخ لرو

د اصطحاک زاویه $\phi = 25^\circ$ د خاوری فی واحد حجم وزن $\gamma = 16.5 \frac{KN}{m^3}$

د خاوری چسپش ضریب $C = 20 \frac{KN}{m^2}$ د مصوونیت ضریب $SF=3$

د تھداب عمق $D_f=1.5m$

په تھداب باندی دخاوری مجازی فشار (q_{all}) او په تھداب باندی د خاوری لخوا مجموعی قوی (Q) پیدا کری .

$$q_u = c' N_c F_{cs} F_{cd} F_{ci} + q N_q F_{qs} F_{qd} F_{qt} + \frac{1}{2} \gamma B N_\gamma F_{\gamma s} F_{\gamma d} F_{\gamma t} \quad \text{حل:}$$

کله چې د اصطحاک زاویه $\phi = 25^\circ$ وی د عمومی جدول نه لرو

Table 3.3 Bearing Capacity Factors

ϕ'	N_c	N_q	N_γ	ϕ'	N_c	N_q	N_γ
20	14.83	6.40	5.39	46	152.10	158.51	330.35
21	15.82	7.07	6.20	47	173.64	187.21	403.67
22	16.88	7.82	7.13	48	199.26	222.31	496.01
23	18.05	8.66	8.20	49	229.93	265.51	613.16
24	19.32	9.60	9.44	50	266.89	319.07	762.89
25	20.72	10.66	10.88				

$$N_c=20.72, N_q=10.66, N_\gamma=10.88$$

قوی عمودی دی نو د قوی زاویه ضرایب به مساوی د 1 سره وی

او د تھداب عمق او د تھداب شکل ضرایب $F_{ci} = F_{qt} = F_{\gamma t} = 1$ Since the load is vertical

ترتیب کوونکی:- انجینر عبدالحمید (رسولی)
مضمون: تہداب ڈیزاین

1: اول باید $\frac{D_f}{B}$ چک کرو

$$\frac{D_f}{B} = \frac{1.5}{2} = 0.75 < 1$$

نوٹ: تہداب شرایط د دوہم حالت لری نو لرو

1: د تہداب د شکل ضرایب

$$F_{cs} = 1 + \left(\frac{B}{L}\right)\left(\frac{N_q}{N_c}\right) = 1 + \left(\frac{2}{2}\right)\left(\frac{10.66}{20.72}\right) = 1.514$$

$$F_{qs} = 1 + \left(\frac{B}{L}\right) \tan\phi' = 1 + \left(\frac{2}{2}\right) \tan 25 = 1.466$$

$$F_{\gamma s} = 1 - 0.4\left(\frac{B}{L}\right) = 1 - 0.4\left(\frac{2}{2}\right) = 0.6$$

1: د تہداب د عمق ضرایب

$$F_{qd} = 1 + 2 \tan\phi' (1 - \sin\phi')^2 \left(\frac{D_f}{B}\right) = 1 + (2) (\tan 25) (1 - \sin 25)^2 \left(\frac{1.5}{2}\right) = 1.233$$

$$F_{cd} = F_{qd} - \frac{1 - F_{qd}}{N_c \tan\phi'} = 1.233 - \left[\frac{1 - 1.233}{(20.72) (\tan 25)} \right] = 1.257$$

$$F_{\gamma d} = 1$$

ضرایب پیدا شول اوس یی پہ لاندی عمومی معادلہ کی وضع کوی

$$q_u = c' N_c F_{cs} F_{cd} F_{ci} + q N_q F_{qs} F_{qd} F_{qt} + \frac{1}{2} \gamma B N_\gamma F_{\gamma s} F_{\gamma d} F_{\gamma t}$$

$$q_u = (20)(20.72)(1.514)(1.257)(1) + (1.5 \times 16.5)(10.66)(1.466)(1.233)(1)$$

$$+ \frac{1}{2}(16.5)(2)(10.88)(0.6)(1)(1) = 788.6 + 476.9 + 107.7 = 1373.2 \text{ kN/m}^2$$

$$q_{\text{all}} = \frac{q_u}{\text{FS}} = \frac{1373.2}{3} = 457.7 \text{ kN/m}^2$$

$$Q = (457.7)(2 \times 2) = \mathbf{1830.8 \text{ kN}}$$

درېم فصل Design of footing

د تهاداب ډيزاين

تهداب د ساختمان يو خاص او اساس برخه دی چه د لاندنی موخي لپاره استفاده کیری

د تهاداب موخي (مقاصد)

- 1:- د تهاداب موخه داده چي د ساختمان بار په يو پراخه سيمه باندي وويشل شي تر څو بار په يوه نقطه باندي عمل ونکړی ، يعنی په خاوری د زيات بار له واریدو څخه وژغوري.
- 2:- د تهاداب دويمه موخه داده چي د ساختمان بار په مساوي شکل وويشل شي او د غير منظمي کهناستنی(نشست) مخنيوي وکړي.
- 3:-تهداب د ساختمان لپاره د ليول سطحه برابروي
- 4:-په ځمکه کي د تهاداب په ننوتولو سره ساختمان ټيکاو(ثبات) اخلي او له تاويدو څخه يي مخنيوی کیری.

د تهاداب ډولونه

تهداب په عمومي ډول په دو برخه وويشل شوی دی.

1 :- ژور تهادابونه

2:- سطحي تهادابونه

1:- ژور تهادابونه

کله چي تهاداب د ساختمان د Super Structure له ټولو څخه بنسکتنی نقطې څخه لاندي واقع شي ژور تهاداب بلل کيري ، Pile , pier , well د ژور تهادابونو په ډله کي راځي

2:- سطحي تهادابونه

ترتيب كوني:- انجينر عبدالحميد (رسولي)
مضمون: د تهاداب ډيزاين

كله چي تهاداب د څخه وروسته متصل واقع شي د سطحي تهادابونو په نوم ياديږي سطحي تهادابونه لاندې ډولونه لري.

1:- غزیدلی یا جداگانه تهادابونه (Spread/single Footing)

2:- مرکب یا مسلسل تهادابونه (Combined/Continuous Footing)

3:- فرشی تهادابونه (Raft/Mat Footing)

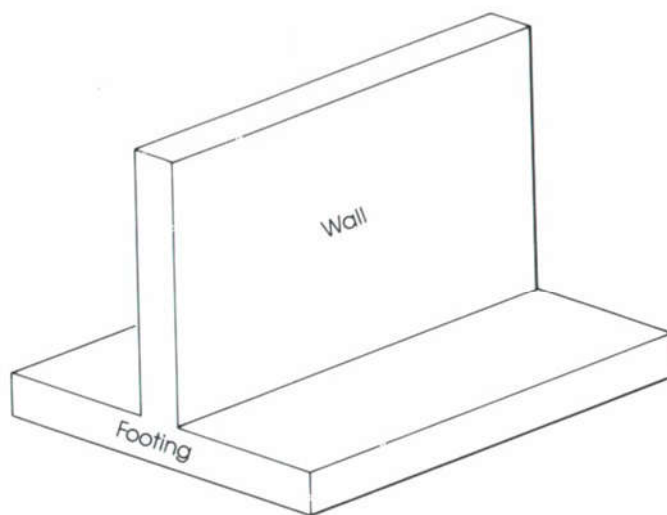
د شکل له مخي تهادابونه

- 1:- مسلسل یا دیوالي تهادابونه 2:- مربعوي تهادابونه 3:- مایل مربعوي تهادابونه
- 4:- پته يي مربعوي تهادابونه 5:- مستطیل شکله تهادابونه 6:- مایل مستطیل شکله تهادابونه

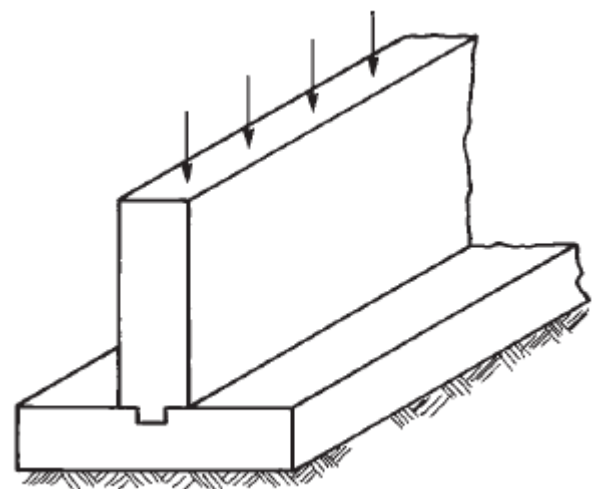
1:- دیوالي تهادابونه

دیوالي تهادابونه د ساختمان بار برداری یوالونو او غیر بار برداری دیوالونو لپاره استفادی کيږی. او همچنان د استنادی دیوالونو لپاره هم کارول کيږی.

Wall footings are used to support structural walls that carry loads for other floors or to support nonstructural walls.



Wall footing.

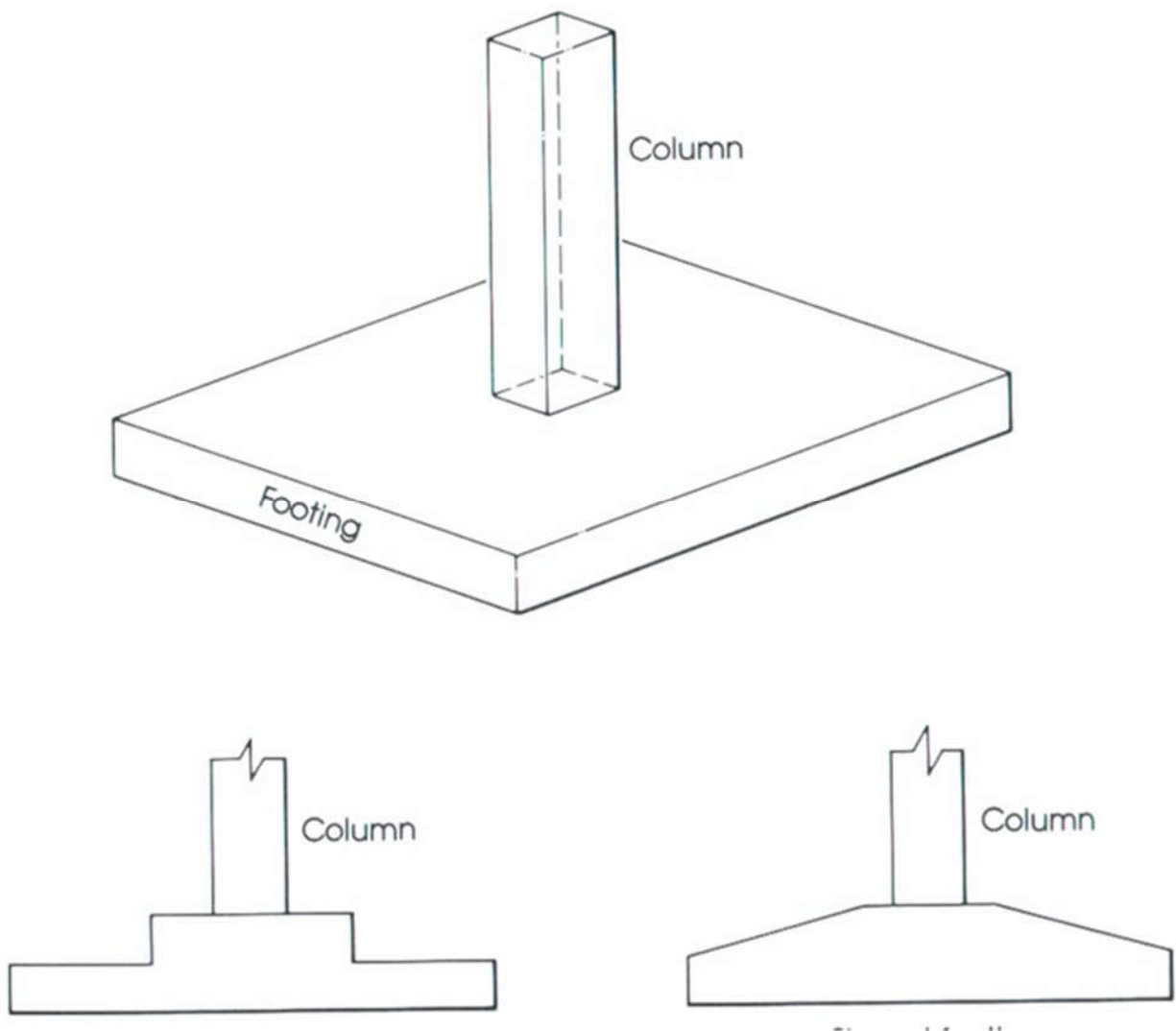


(a) Strip or wall footing.

2:- جداگاني تهادابونه

جداگانه تهادابونه د هر کالم (پايه) لپاره يو تهاداب استعمالیږي، او يا د پایو تر منځی فاصله چې زیاتی وی استفادی کیری او تر ټول اقتصادي ترین تهادابونه دی چه نن سبا ډیر زیات کاروال کیری.

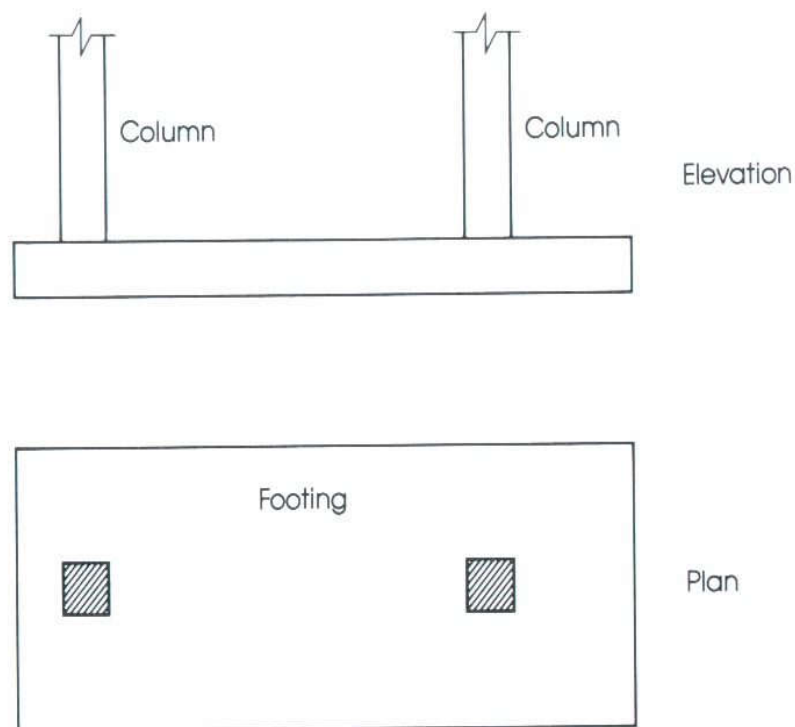
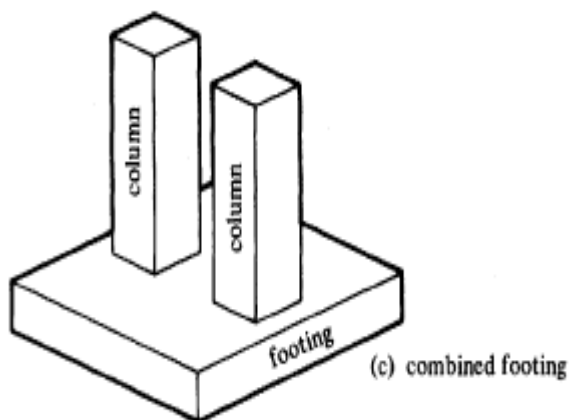
Isolated or single footings are used to support single columns. This is one of the most economical types of footings and is used when columns are spaced at relatively long distances



3:- مرکب تهادابونه

مرکب تهادابونه معمولا د دوه کالمونه، يا درې ستنې چې په يو قطار کې وي. او همچنان مرکب تهادابونه هغه وخت کارول کيږي کله چې دوه تهادابونه ډير نږدې وي يا کله چې يو کالم د ملکیت په اخير کرښې ته نږدې وي

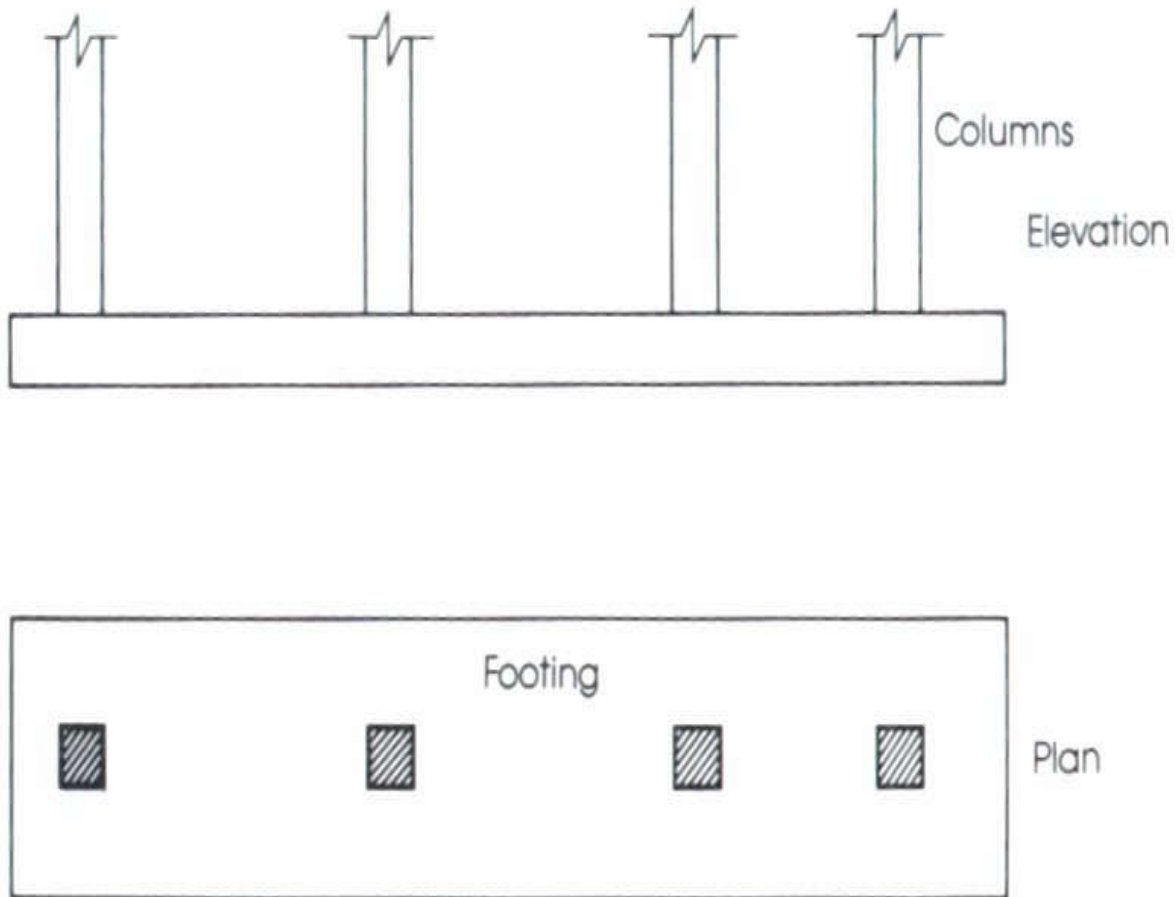
Combined footings usually support two columns, or three columns in a row. Combined footings are used when two columns are so close that single footings cannot be used or when one column is located at or near a property line.



4:- مسلسل تھدابونہ

مسلسل تھدابونہ د دریو یا زیات تر دریو پایو لپارہ استفادی کیری ، مسلسل تھدابونہ د یو وایہ
تولو پایو لاندی پہ معین عرض سرہ استمعالییری

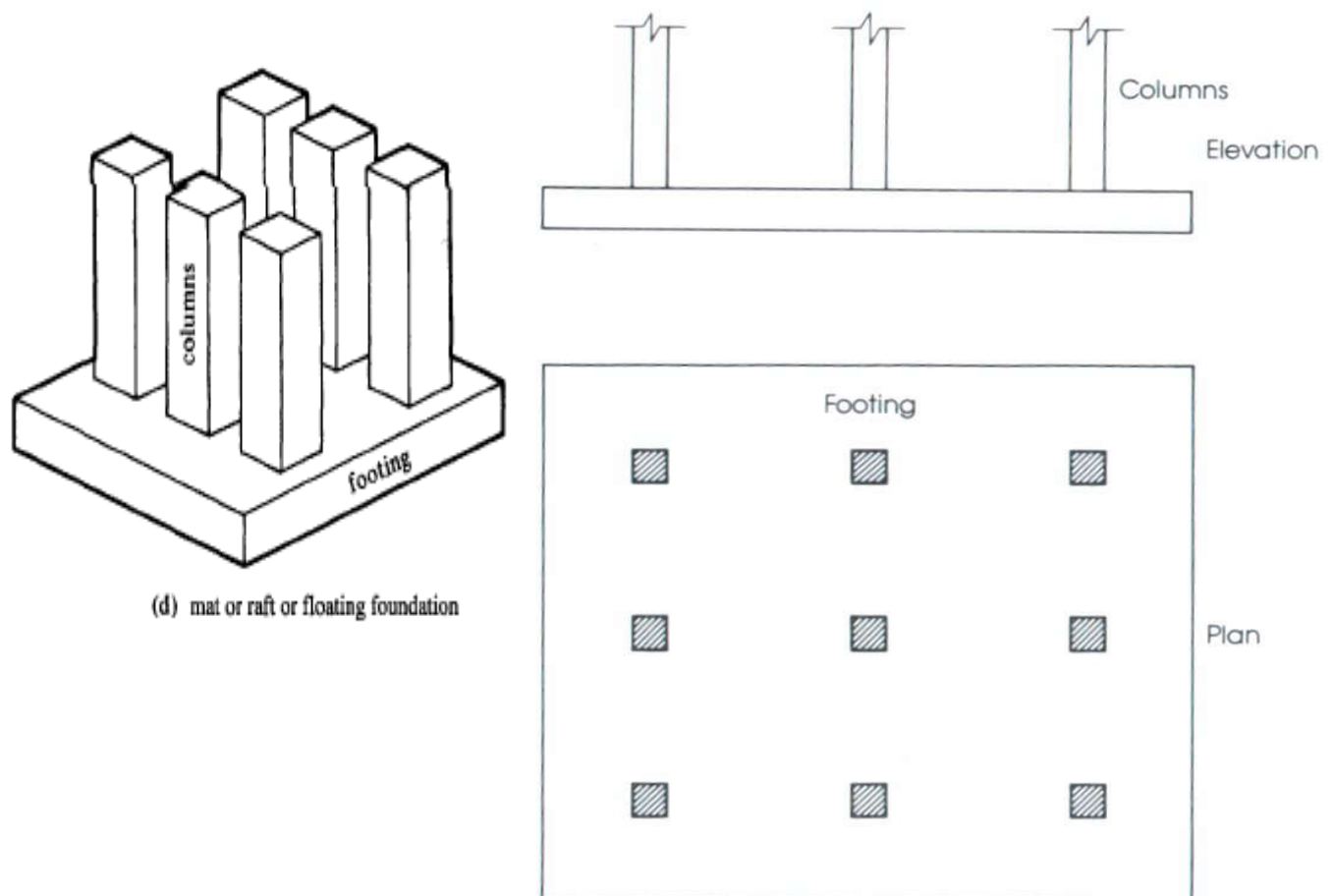
Continuous footings support a row of three or more columns. They have limited width and continue under all columns.



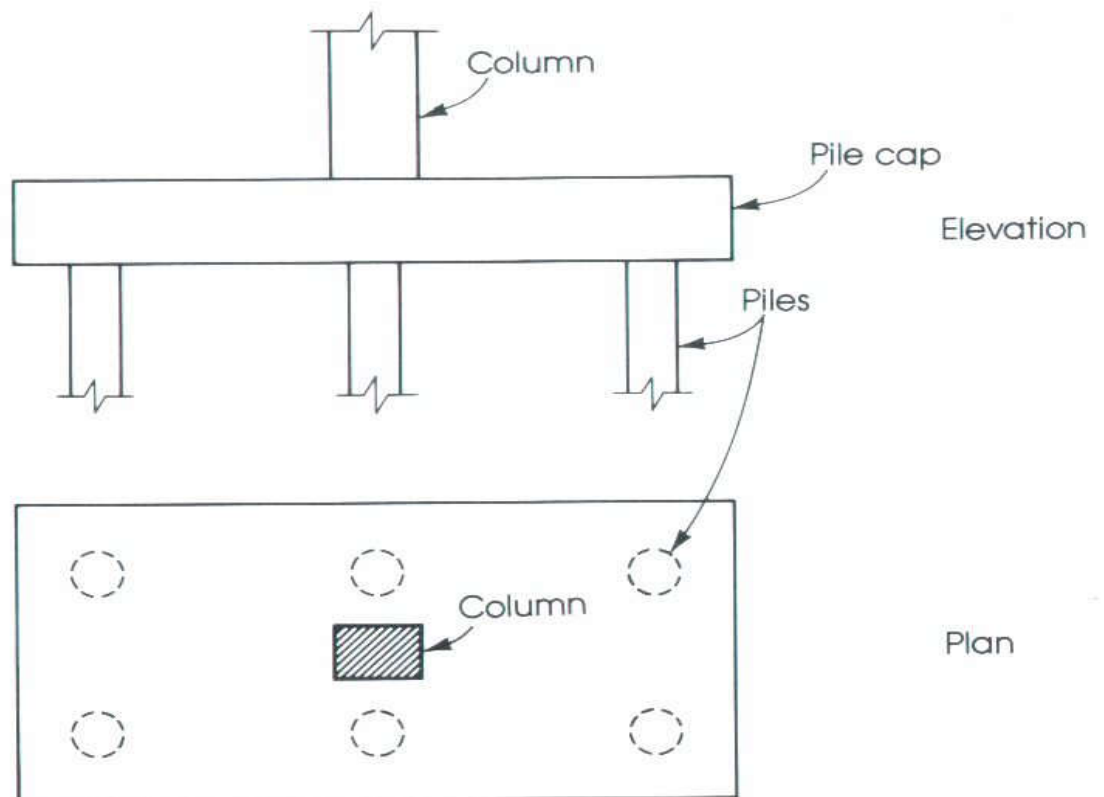
5:- فرشہ تہدابونہ

فرشہ تہداب د ساختمان ٹولو پایو لاندی پہ شکل د فرش استعمالیری ، دغہ تہداب پہ ہغہ وخت کی استفادی کیری چہ د خاوری برداشت قابلیت ڈیر کمزوری وی یعنی جداگانی تہدابونو مساحت یو د بل سر تکر کوی

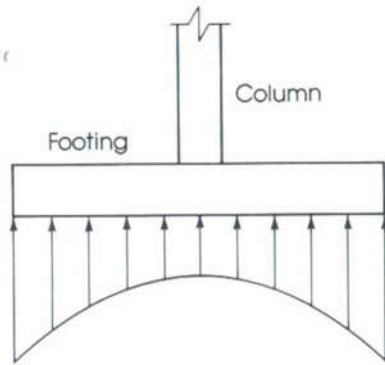
Rafted or mat foundation consists of one footing usually placed under the entire building area. They are used, when soil bearing capacity is low, column loads are heavy single footings cannot be used



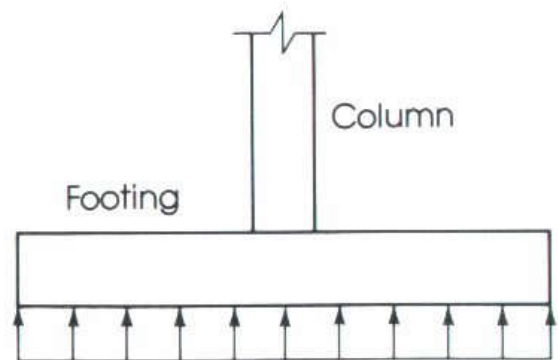
Pile caps are thick slabs used to tie a group of piles together to support and transmit column loads to the piles.



د تھداب لاندی د خاوری عکس العمل نظر د خاوری خواص ته



Soil pressure distribution in cohesive soil.



Soil pressure distribution in cohesionless soil.

مثال:- دپایي لاندی تھداب ڈیزاین کری چه د پایي ابعاد $18*18in$ دی او د پایي لخوا نهایی بار $P_u=300.54Kip$ ، خالص بار $P=235.64 kip$ عمل کری وی ، او همچنان د خاوری برداشت قابلیت $q_a=2.204kip/ft^2$ وی ، او د ډکونکو خاورو حجمی وزن $=0.1Klb/ft^3$ او د کانکریټو حجمی وزن $\gamma_{fill}=0.15kip/ft^3$ وی ، نظر جیوتکنیکل راپورته د تھداب ارتفاع $18in$ وی ، نوموړی تھداب د ځمکی له سطحه څخه په $5ft$ کی لاندی قرار لری ، دا چې د کانکریټو فشاری مقاومت $f_c=3ksi$ وی او د فولادی سخانو کشش مقاومت $f_y=40ksi$ وی

حل:-

راکړ شوی ارقام (Given Data)

$$\begin{array}{ll}
 P_u=300.54Kip & f_c=3ksi \\
 P=235.64 Kip & f_y=40ksi \\
 q_a=2.204kip/ft^2 & D_f=5ft \\
 \gamma_{fill}=0.1kip/ft^3 & h=18in \\
 \gamma_c=0.15kip/ft^3 & \text{Column size}=18*18in
 \end{array}$$

Step-1

د خاوری خالص فشار

$$\begin{aligned}
 q_e &= q_a - w_s \\
 w_s &= \gamma_{fill}(D_f - h) + \gamma_c * h = 0.1(5 - 1.5) + 0.15 * 0.15 \\
 w_s &= 0.575 \\
 q_e &= q_a - w_s = 2.204 - 0.575 = 1.629kip/ft^2
 \end{aligned}$$

$$A = \frac{p}{q_e} = \frac{235.64}{1.624} = 144.63 ft^2 \approx 144 ft^2$$

که چیری مربع شکل تھداب وی نو لروی

$$B = L \text{ and } B \times B \text{ so } B^2 = 144 ft^2 \Rightarrow \sqrt{B} = \sqrt{144}$$

$$B = 12 ft$$

Step-2

په دوهم مرحله کی دو طرفه برش په لاس راوری (Two way shear or punching shear)

$$V_u = P_u - q_a (l_c + d_{average}) * (b_c + d_{average})$$

$$q_a = \frac{P_u}{A} = \frac{300.54}{144} = 2.087 \frac{kip}{ft^2}$$

$$d_{average} = h - 3 - \text{one bar dia}$$

if we use #8 so $d_{average} = 14 in$

$$V_u = 300.54 - 2.087 \left(1.5 + \frac{14}{12}\right) \left(1.5 + \frac{14}{12}\right) = 285.69 kip$$

اوس مقاومه عرضانی قوه په لاس راوری چي عبارت دی له

$$\phi V_c = 0.75 \cdot 4 \cdot \sqrt{f_c} \cdot b_0 \cdot d_{average}$$

$$b_0 = 2(l_c + b_c + 2d) \text{ or } 4(b_c + d_{average}) = 4 \left(1.5 + \frac{14}{12}\right) = 10.67 ft = 128 in$$

$$\phi V_c = 0.75 \cdot 4 \cdot \sqrt{3000} \cdot 128 \cdot 14 = \frac{294455.64}{1000} = 294.45 kip$$

خرنگه چي زمونږ $V_u < \phi V_c$ زیات دی نو d درست انتخاب شوی یا د تھداب ارتفاع (h) صحیح دی

اوس یو طرفہ (یو لوریزہ) اعظمی عرضانی قوہ پہ لاس راورو

1:- د لوری سرہ اعظمی عرضانی قوہ پہ لاس راورو

$$V_{u-B} = q_a \left(\frac{l}{2} d_{average} \right) B = 2.087 \left(\frac{5}{2} \frac{1.5}{12} \right) 12 = 14.609 \text{kip}$$

اوس د لوری سرہ موازہ مقاومہ عرضانی قوہ پہ لاس راورو

$$\phi V_c = 0.75 \cdot 2 \cdot \sqrt{f_c} \cdot B \cdot d_{average}$$

$$\phi V_c = 0.75 \cdot 4 \cdot \sqrt{3000} \cdot 144 \cdot 14 = \frac{331262.6}{1000} = 331.263 \text{kip}$$

خرنگہ چي L او B لوری سرہ مساوی دی یعنی تھداب مربع شکل دی نو د بل لوری محاسبہ نہ صرف نظر کیڑی.

Step-3

اوس مومنت پہ لاس راورو

$$M_{u-B} = q_a \left(\frac{(L \cdot l_c)}{8} \right) B = 2.087 \left(\frac{(5 \cdot 1.5)}{8} \right) 12 = 38.34 \text{kip ft}$$

$$M_{u-B} = 480.18 \text{kip in}$$

$$M_{u-h} = \frac{M_{u-B}}{\phi} = \frac{480.18 \text{kip in}}{0.9} = 511.315 \text{kip in}$$

$$R_{h-B} = \frac{M_{h-B}}{B \cdot d_{average}^2} = \frac{511.315 \text{kip in}}{12 \cdot 12 \cdot 14^2} = 18.11 \text{PSI}$$

$$\rho_B = 0.85 \frac{f_c}{f_y} \left[1 + \sqrt{\left(1 + \frac{2R_h}{0.85f_c} \right)} \right] = 0.85 \frac{3}{40} \left[1 + \sqrt{\left(1 + \frac{2 \cdot 18.11}{0.85 \cdot 3000} \right)} \right]$$

$$\rho_B = 0.000454$$

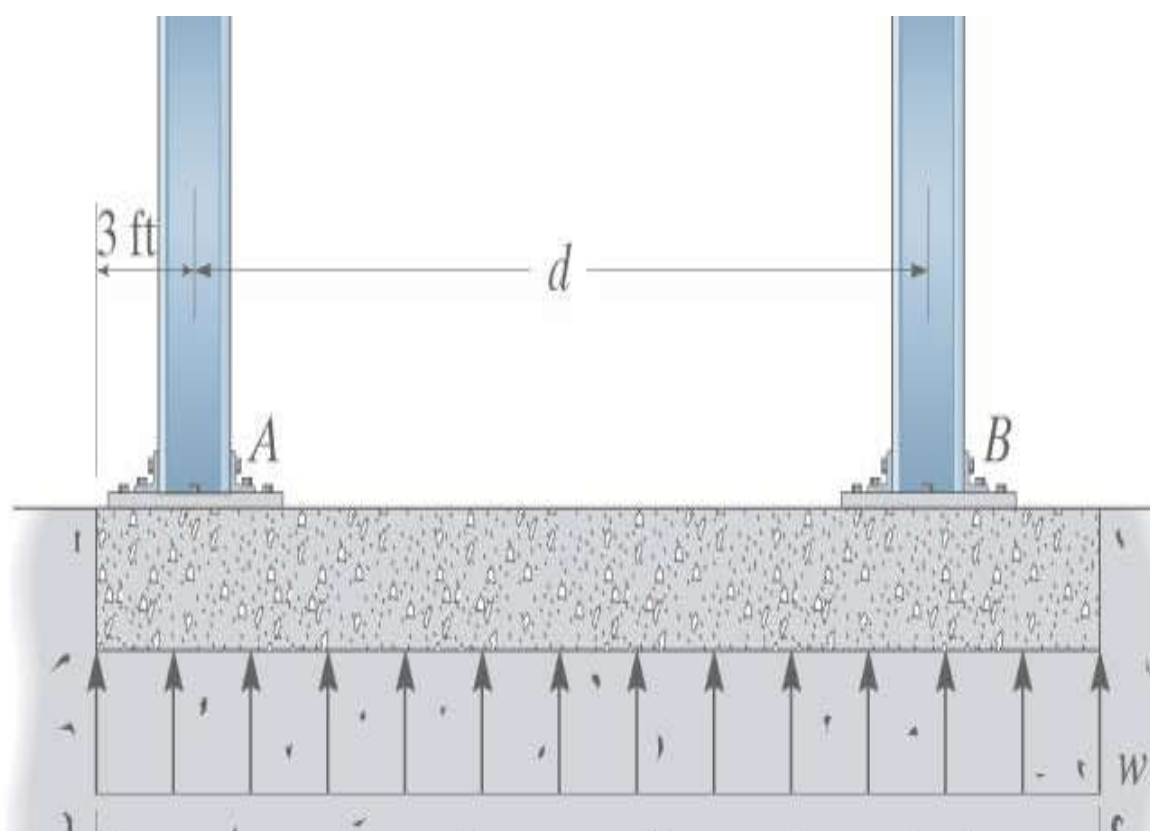
چون رو $\rho_B < \rho_{min}$ لوی دی بنا ρ_{min} استفادی کوی

$$A_{S_{min}} = \rho_{min} B d_{average} = 0.002 \quad 12 \quad 12 \quad 14 = 4.032 in^2$$

Footing Design

Part II

Combined footing



مركب تهادابونه

مركب تهادابونه د لاندي اړتياو له مخي ټاكل كيږي

1. كله چې دو پايي دومره نژدي وي چې د دو تهادابونو مساحت يو د بل سره ټكړي وكړي
2. كله چې د يوي دواني باندي پايي داسي وي چې د بل چاه تعمير ته ډير نژدي وي
3. كله چې د تهاداب د خاوري برداشت قوه كمزوري وي

د مركب تهادابونو ډولونه

- مستطيلي تهادابونه
- ذونقه يي تهادابونه
- كنسولي تهادابونه
- پته يي تهادابونه

د مستطيلي مركب تهاداب د ډيزاين لپاره لاندي ټكي په پام كي ونيول شي

په پايه باندي وارد شوي مجموعي بار معلومول ❖

د تهاداب تقريبي مساحت ټاكل ❖

د تهاداب د ابعادو ټاكل او د هغي له مخي د خاوري فشار پيدا كول ❖

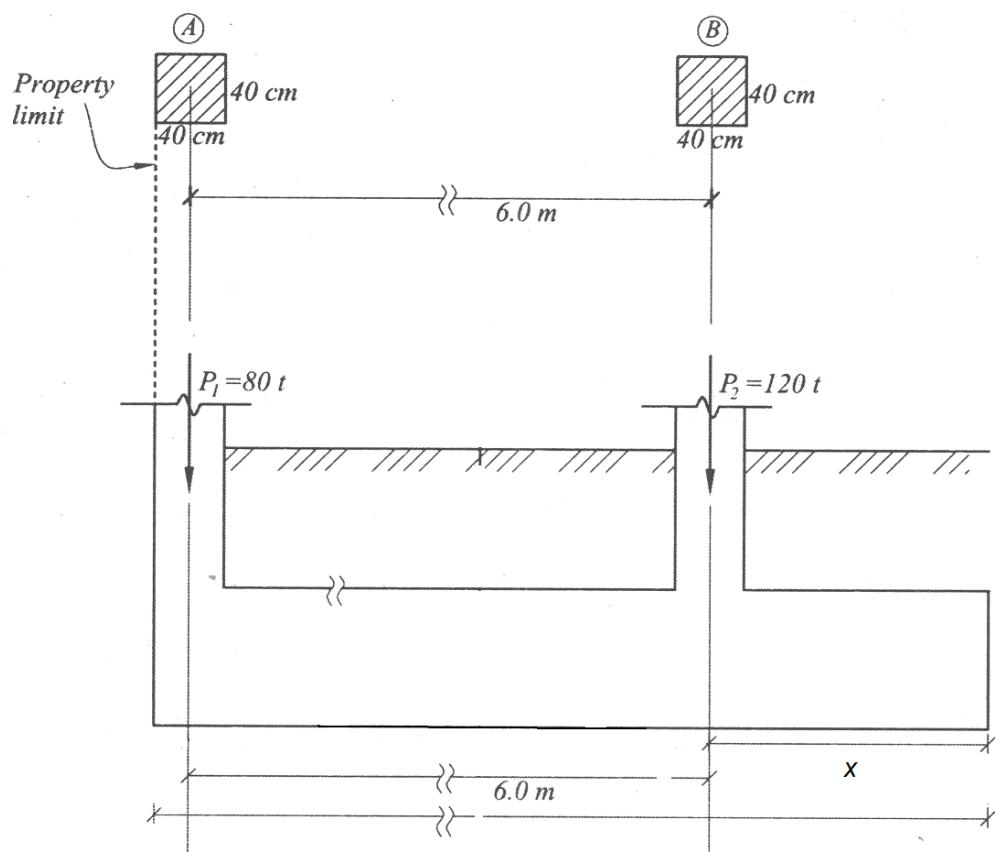
په مختلفو مقطعو كي د عرضي قوي پيدا كول ❖

مومنټ پيدا كول ❖

مثال: د C_A او C_B پایو لپاره مرکب تھداب ډیزاین کړی چې د پایو د مرکز نه تر مرکز فاصله او 6m ، $40 \times 40\text{cm}$ ابعاد لری چې په ترتیب سره 800KN په C_A باند او 120KN په C_B باندی او د خاوری د برداشت قابلیت $q_a = 200\text{Kpa}$ ، او د کانکریتو فشار مقاومت $200 = f_c \text{ N/mm}^2$ ، او د سیخانو کشش مقاومت $f_y = 420 \text{ N/mm}^2$ چی په لاندی شکل که خودل شوی

Design a combined footing As shown

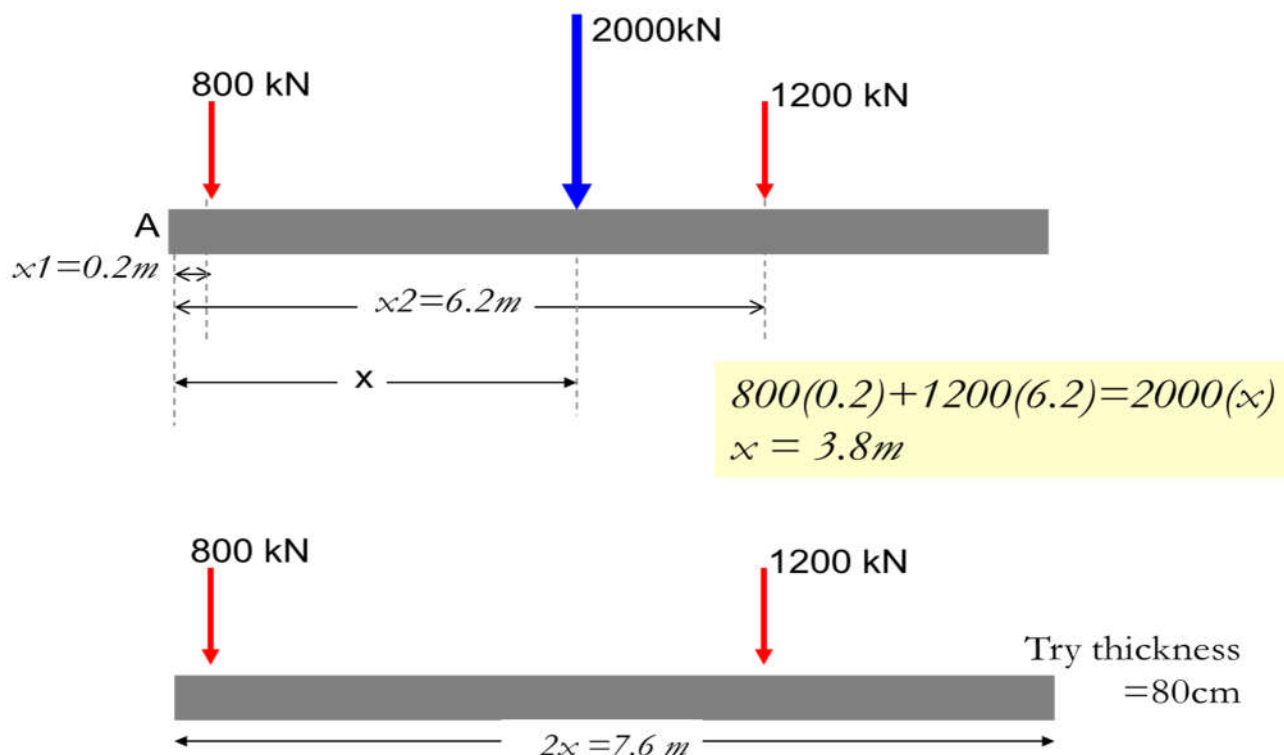
$$q_{all(net)} = 20t / m^2 = 200kPa \quad f'_c = 25 \text{ N/mm}^2 \quad f_y = 420 \text{ N/mm}^2$$



د مجموعی قوه فاصله پیدا کوی

Dimension calculation

The base dimension to get uniform distributed load



Area required

$$q_{all(net)} = 20t / m^2 = 200 kPa ,$$

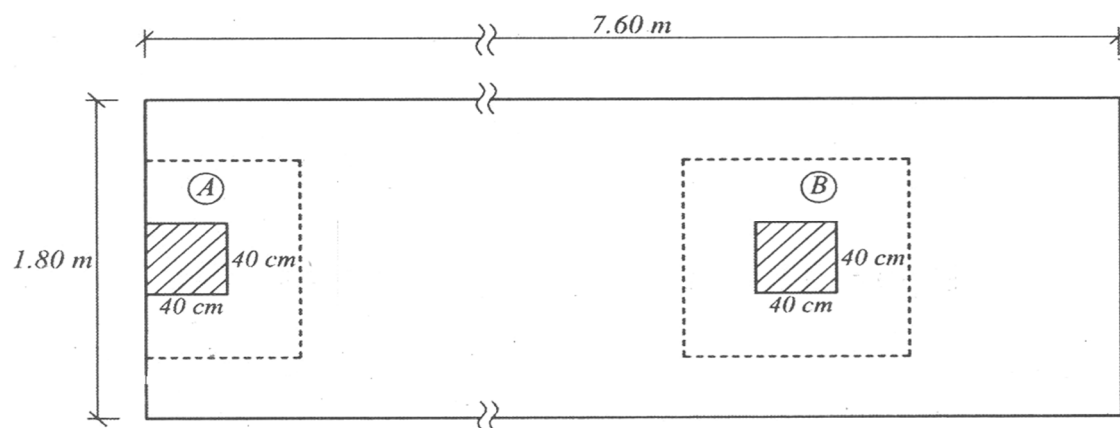
$$P_u = 1.3(P_s) = 1.3(2000) = 2600 kN$$

$$A_g = \frac{P_s}{q_{all(net)}} = \frac{2000 \times 10^3}{200 \times 10^3} = 10 m^2 \approx 7.6 * 1.8$$

$$q_u = \frac{P_u}{A} = \frac{(2600) \times 10^3}{7.6 * 1.8} = 190 \times 10^3 Pa = 190 kPa$$

د دو طرفه برش پیدا کول Check for punching Shear

توت: فرض کوی چي $d = 730 \text{ mm}$



Column A

$$b_o = 2(765) + 1130 = 2260 \text{ mm}$$

$$\phi V_c = \phi \frac{\sqrt{f'_c}}{3} b_o d = 0.75 \times \frac{\sqrt{25}}{3} \times 730 \times 2260 / 1000 = 2062.3 \text{ kN}$$

$$\phi V_c = \phi \left(2 + \frac{\alpha_s d}{b} \right) \frac{\sqrt{f'_c}}{12} b_o d = 0.75 \left(2 + \frac{30 \times 730}{2260} \right) \times \frac{\sqrt{25}}{12} \times 730 \times 2260 / 1000 = 6027 \text{ kN}$$

$$V_u = 800(1.3) - 1.13 \times 0.765 \times 190 = 875.8 \text{ kN} < \phi V_c \quad \text{oK}$$

Column B

$$b_o = 4[(730 + 400)] = 4520 \text{ mm}$$

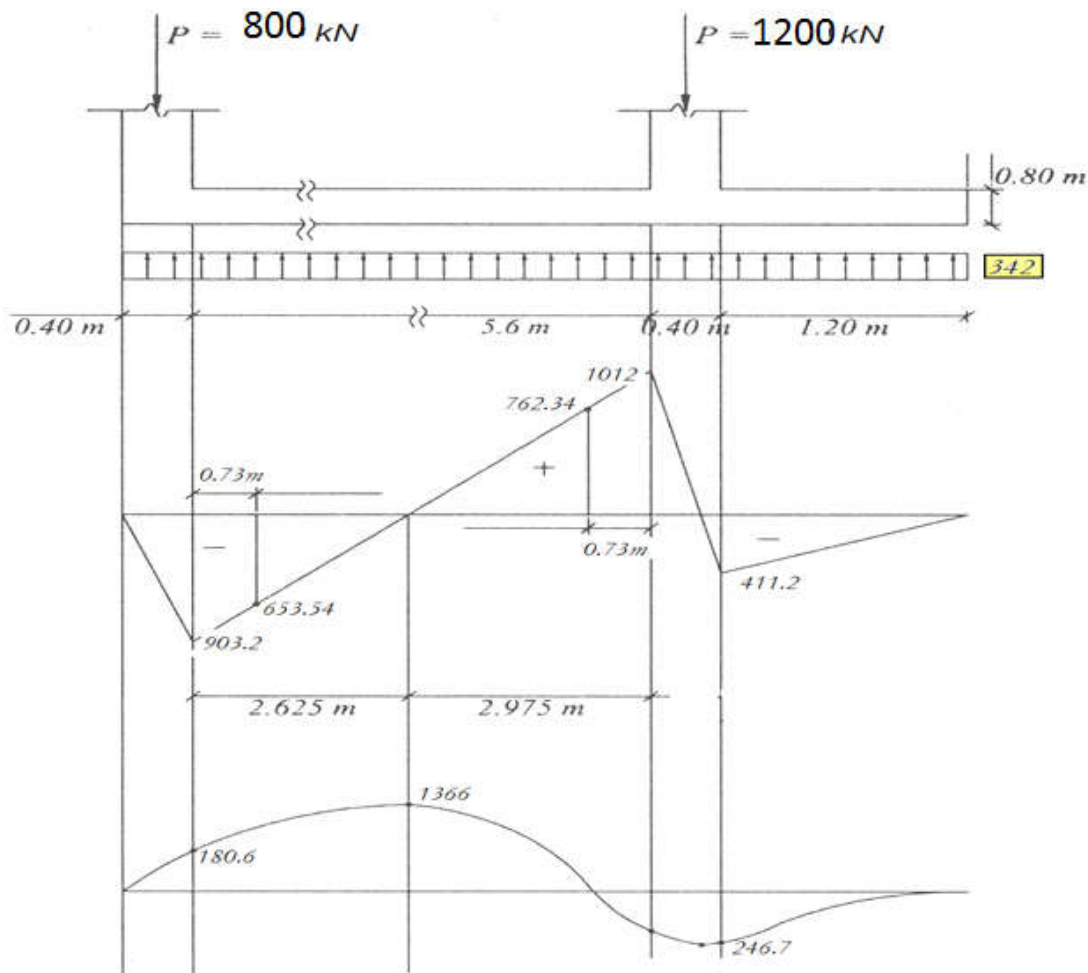
$$\phi V_c = \phi \frac{\sqrt{f'_c}}{3} b_o d = 0.75 \times \frac{\sqrt{25}}{3} \times 730 \times 4520 / 1000 = 4124.4 \text{ kN}$$

$$\phi V_c = \phi \left(2 + \frac{\alpha_s d}{b} \right) \frac{\sqrt{f'_c}}{12} b_o d = 0.75 \left(2 + \frac{40 \times 730}{4520} \right) \times \frac{\sqrt{25}}{12} \times 730 \times 4520 / 1000 = 13322.5 \text{ kN}$$

$$V_u = 1200(1.3) - 1.13 \times 1.13 \times 190 = 1317.4 \text{ kN} < \phi V_c \quad \text{oK}$$

Draw S.F.D & B.M.D

د خاوری فشار *Stress under footing*
 $= 190 * 1.8 = 342 \text{ kN/m}$



Check for beam shear

$$b = 1800\text{mm}, d = 730\text{mm}$$

$$\phi V_C = 0.75 \times \frac{\sqrt{25}}{6} \times 730 \times 1800 / 1000 = 821.25\text{kN}$$

$$\text{Max.} \rightarrow V_U \text{ at } \underline{d} \text{ from column face} = 762.34\text{kN}$$

$$V_U < \phi V_C$$

مومنت پہ طولی سیکشن کی Bending moment Long direction

$$-ve M = 1366\text{kN.m}$$

$$b = 1800\text{mm}, d = 730\text{mm}$$

$$\rho = 0.85 * \frac{25}{420} \left[1 - \sqrt{1 - \frac{2 \times 10^6 * 1366}{0.9(0.85)25 * 730^2 * 1800}} \right] = 0.0039$$

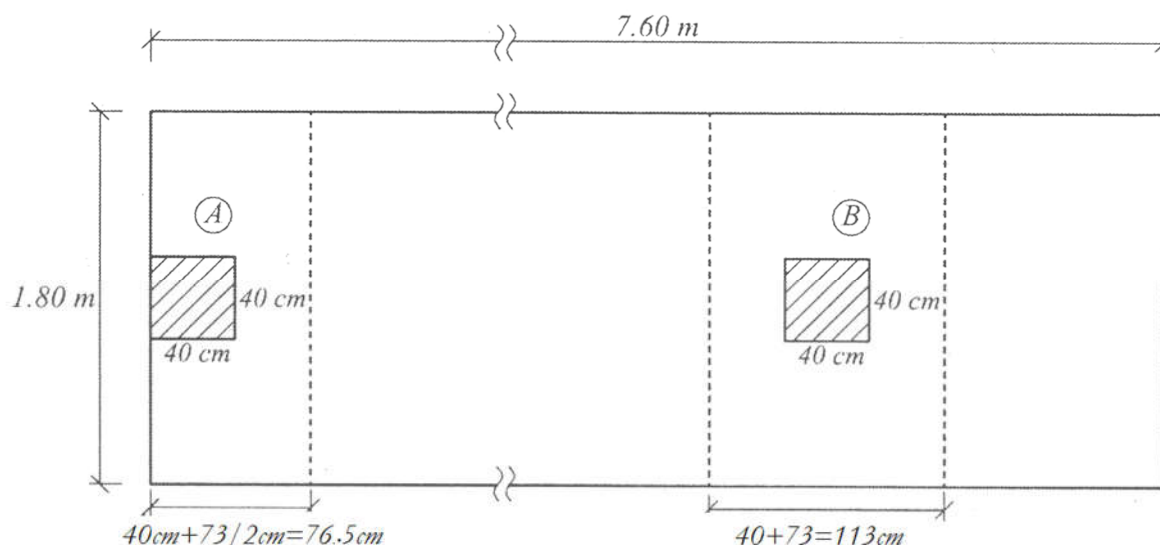
$$A_S = 0.0039 \times 730 \times 1000 = 2847\text{mm}^2 = 28.5\text{cm}^2 \text{ use } 9\phi 20 / m \text{ Top}$$

$$+ve M = 246.7\text{kN.m}$$

$$b = 1800\text{mm}, d = 730\text{mm}$$

$$\rho = 0.85 * \frac{25}{420} \left[1 - \sqrt{1 - \frac{2 \times 10^6 * 246.7}{0.9(0.85)25 * 730^2 * 1800}} \right] = 0.0007 < \rho_{\min}$$

$$A_{S\min} = 0.0018 \times 800 \times 1000 = 1440\text{mm}^2 = 14.4\text{cm}^2 \text{ use } 7\phi 16 / m \text{ Bottom}$$

مومنت پہ عرضی سیکشن کی Bending moment Short direction**Under Column A**

$$M = \frac{1040}{(1.8 * 0.765)} \times \frac{0.765}{2} \left(\frac{1.8 - 0.4}{2} \right)^2 = 141.6$$

$$b = 765 \text{ mm}, d = 730 \text{ mm}$$

$$\rho = 0.85 * \frac{25}{420} \left[1 - \sqrt{1 - \frac{2 \times 10^6 * 141.6}{0.9(0.85)25 * 730^2 * 765}} \right] < \rho_{\min}$$

$$A_{S \min} = 0.0018 \times 800 \times 765 = 1101.6 \text{ mm}^2 = 11 \text{ cm}^2 \quad \text{use } 7\phi 14 / m$$

Under Column B

$$M = \frac{1560}{(1.8 * 1.13)} \times \frac{1.13}{2} \left(\frac{1.8 - 0.4}{2} \right)^2 = 212.33$$

$$b = 1130 \text{ mm}, d = 730 \text{ mm}$$

$$\rho = 0.85 * \frac{25}{420} \left[1 - \sqrt{1 - \frac{2 \times 10^6 * 212.33}{0.9(0.85)25 * 730^2 * 1130}} \right] < \rho_{\min}$$

$$A_{S \min} = 0.0018 \times 800 \times 765 = 1101.6 \text{ mm}^2 = 11 \text{ cm}^2 \quad \text{use } 7\phi 14 / m$$

Shrinkage Reinforcement in short direction

$$A_{S_{\min}} = 0.0018 \times 800 \times 765 = 1101.6 \text{ mm}^2 = 11 \text{ cm}^2 \quad \text{use } 7\phi 14 / \text{m}$$

Under Column B

$$M = \frac{1560}{(1.8 * 1.13)} \times \frac{1.13}{2} \left(\frac{1.8 - 0.4}{2} \right)^2 = 212.33$$

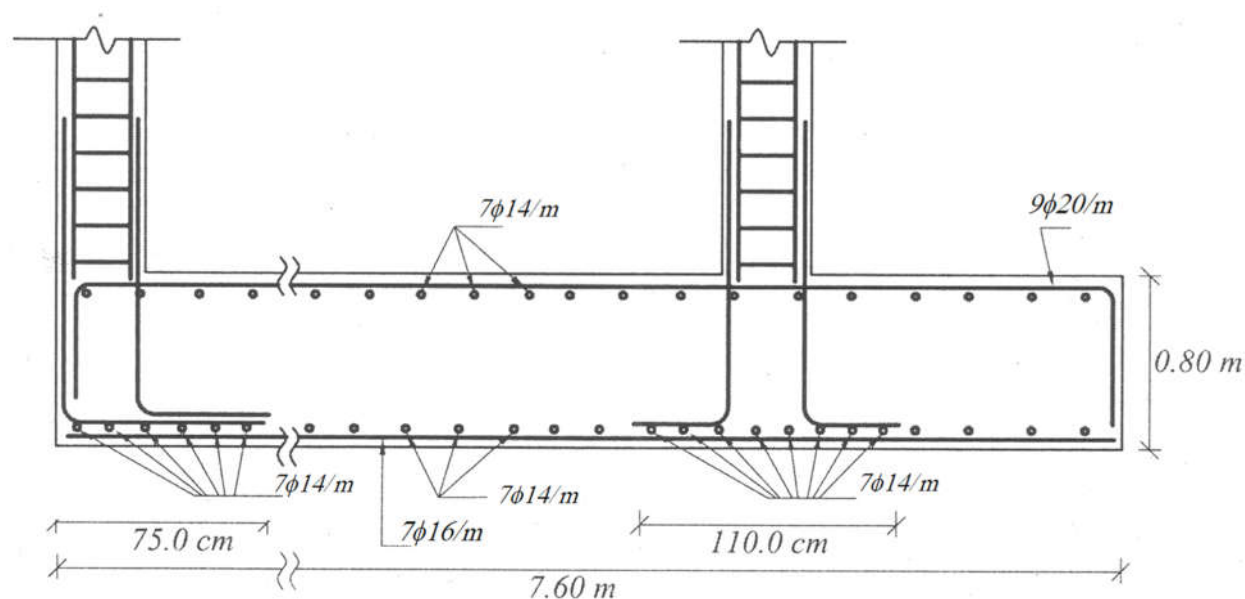
$$b = 1130 \text{ mm}, d = 730 \text{ mm}$$

$$\rho = 0.85 * \frac{25}{420} \left[1 - \sqrt{1 - \frac{2 \times 10^6 * 212.33}{0.9(0.85)25 * 730^2 * 1130}} \right] < \rho_{\min}$$

$$A_{S_{\min}} = 0.0018 \times 800 \times 765 = 1101.6 \text{ mm}^2 = 11 \text{ cm}^2 \quad \text{use } 7\phi 14 / \text{m}$$

Shrinkage Reinforcement in short direction

$$A_{S_{\min}} = 0.0018 \times 800 \times 765 = 1101.6 \text{ mm}^2 = 11 \text{ cm}^2 \quad \text{use } 7\phi 14 / \text{m}$$



Footing Design Part III

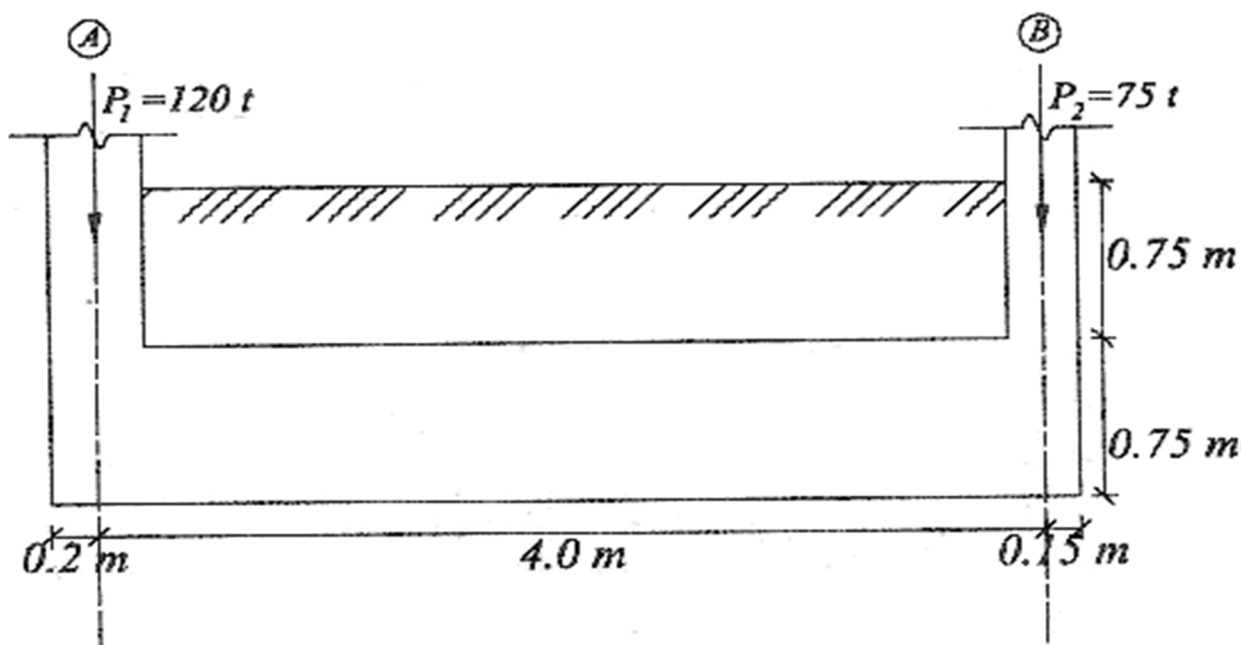
نوٹ: پہ لاندنیو ٹولو مثالونو کہ پورتنی تکرار استفادی کیری

Combined footing, strip footing, & Mat foundation

Example 2

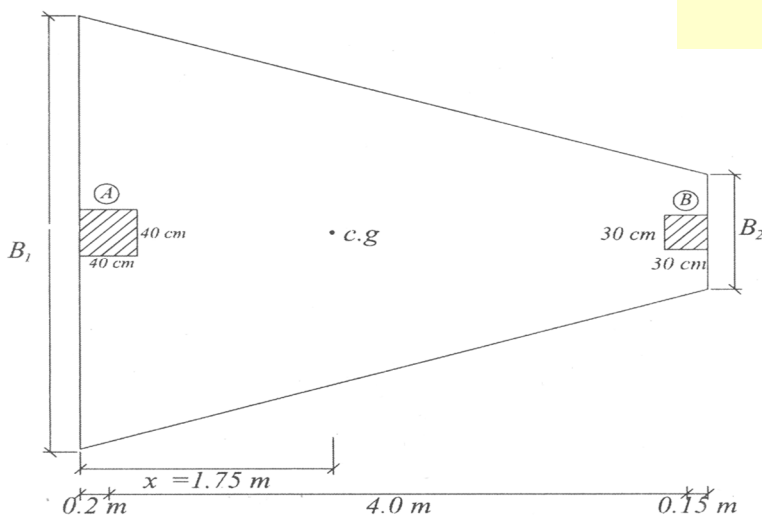
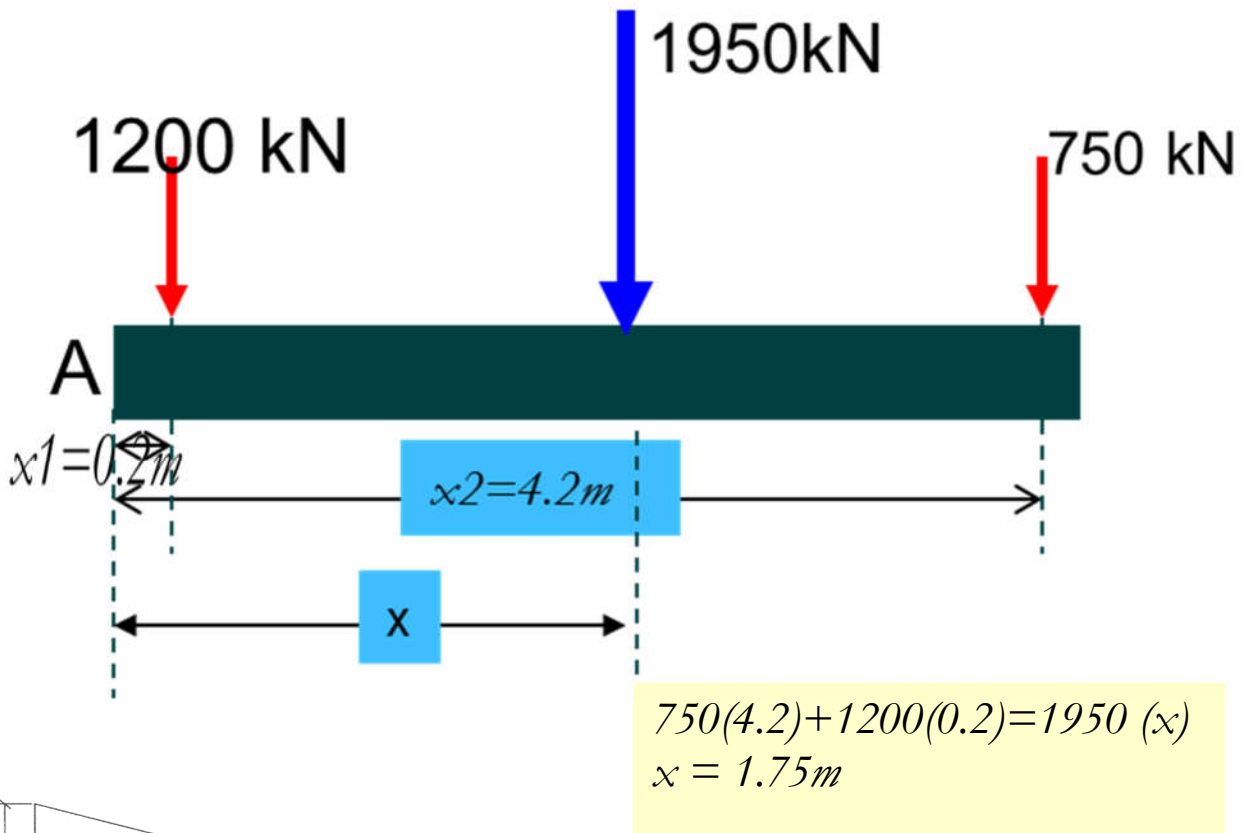
Design a combined footing As shown

$$q_{all(net)} = 18 t / m^2 = 180 kPa \quad f'_c = 25 N/mm^2 \quad f_y = 420 N/mm^2$$



Dimension calculation

The base dimension to get uniform distributed load




$$x = \left(\frac{B_1 + 2B_2}{B_1 + B_2} \right) \frac{L}{3}$$

Area required

$$q_{all (net)} = 20 t / m^2 = 200 kPa ,$$

$$A_g = \frac{P_s}{q_{all (net)}} = \frac{1950 \times 10^3}{180 \times 10^3} = 10.8 m^2$$




$$\left(\frac{B_1 + B_2}{2} \right) L = 10.8$$

$$\left(\frac{B_1 + B_2}{2} \right) 4.35 = 10.8$$

$$\left(\frac{B_1 + B_2}{2} \right) = 2.5$$

$$B_1 + B_2 = 5$$



$$x = \left(\frac{B_1 + 2B_2}{B_1 + B_2} \right) \frac{L}{3} = \left(\frac{5 + B_2}{5} \right) \frac{4.35}{3}$$

$$1.75 = 1.45 + 0.29 B_2$$

$$B_2 = 1 m$$

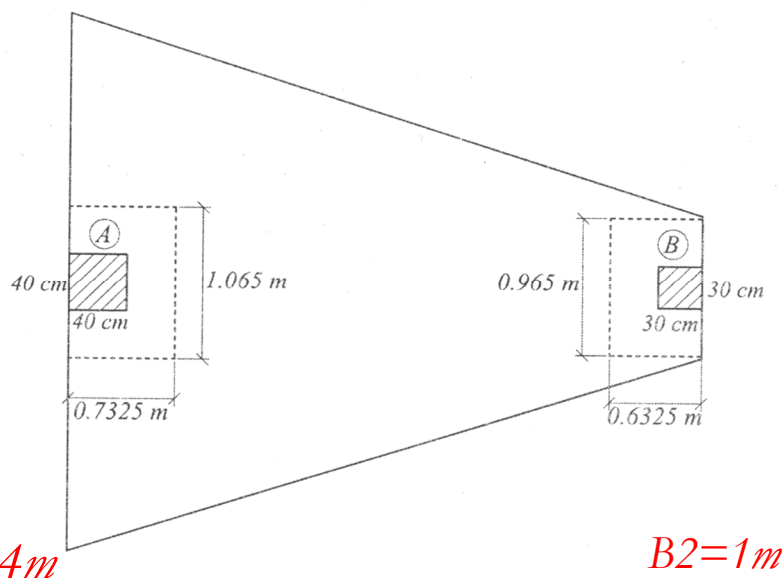
$$B_1 = 4 m$$

$$q_u = \frac{P_u}{A} = \frac{1.3(1950) \times 10^3}{10.8} = 235 \times 10^3 Pa = 235 kPa$$

Check for punching Shear

$$h = 750 \text{ mm}$$

$$d = 732 \text{ mm}$$



Column - A

$$b_o = 2(732) + 1065 = 2590 \text{ mm}$$

$$\phi V_c = \phi \frac{\sqrt{f'_c}}{3} b_o d = 0.75 \times \frac{\sqrt{25}}{3} \times 665 \times 2590 / 1000 = 2160.4 \text{ kN}$$

$$\phi V_c = \phi \left(2 + \frac{\alpha_s d}{b} \right) \frac{\sqrt{f'_c}}{12} b_o d = 0.75 \left(2 + \frac{30 \times 665}{2590} \right) \times \frac{\sqrt{25}}{12} \times 665 \times 2590 / 1000 = 5222 \text{ kN}$$

$$V_u = 1200(1.3) - 1.065 * 0.733 * 235 = 1376.6 \text{ kN} < \phi V_c \quad \text{oK}$$

Column - B

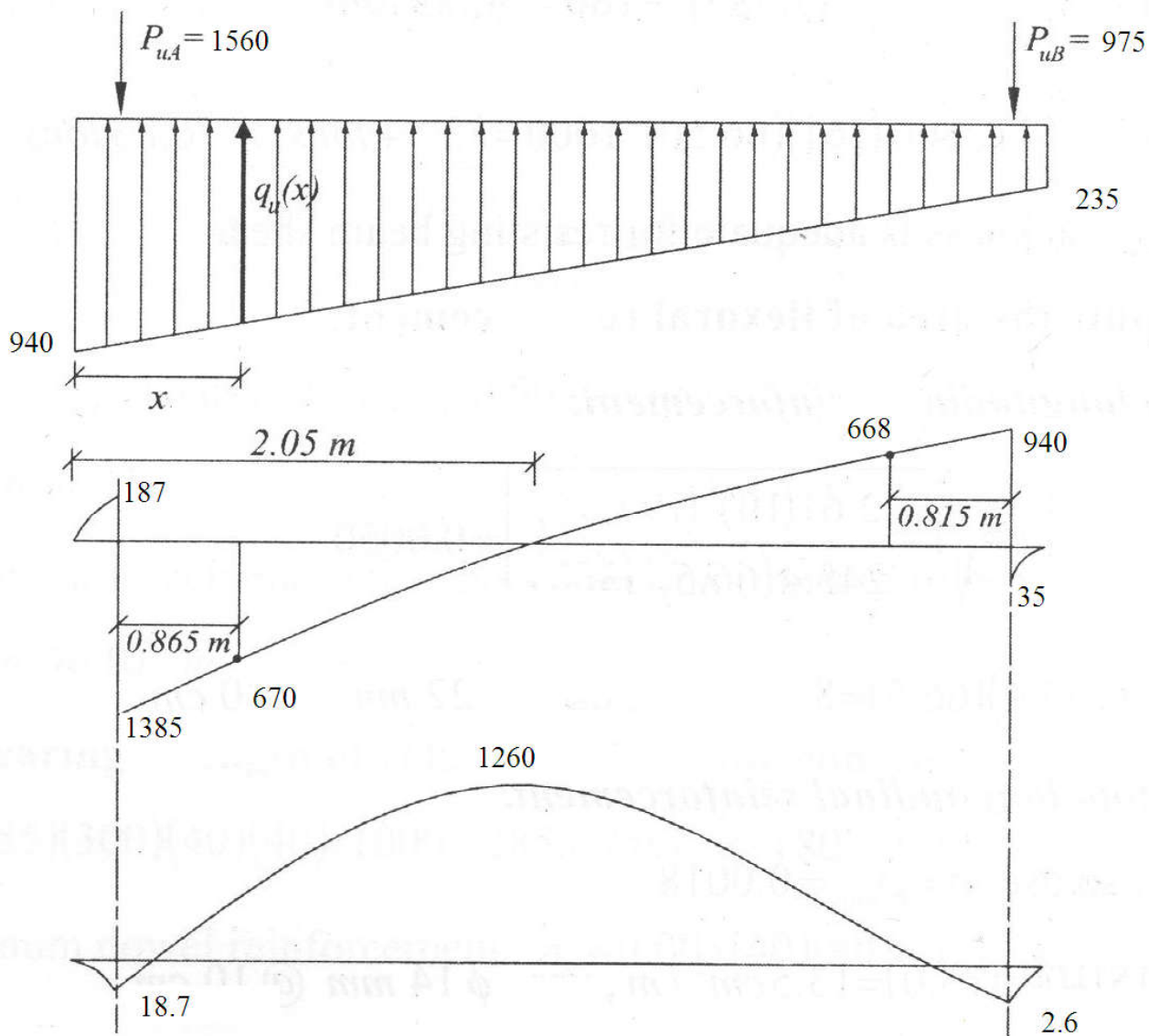
$$b_o = 2(633) + 965 = 2231 \text{ mm}$$

$$\phi V_c = \phi \frac{\sqrt{f'_c}}{3} b_o d = 0.75 \times \frac{\sqrt{25}}{3} \times 665 \times 2231 / 1000 = 1854.5 \text{ kN}$$

$$\phi V_c = \phi \left(2 + \frac{\alpha_s d}{b} \right) \frac{\sqrt{f'_c}}{12} b_o d = 0.75 \left(2 + \frac{30 \times 665}{2231} \right) \times \frac{\sqrt{25}}{12} \times 665 \times 2231 / 1000 = 5273 \text{ kN}$$

$$V_u = 800(1.3) - 0.965 * 0.633 * 235 = 896.5 \text{ kN} < \phi V_c \quad \text{oK}$$

Draw S.F.D & B.M.D



Empirical S.F.D & B.M.D

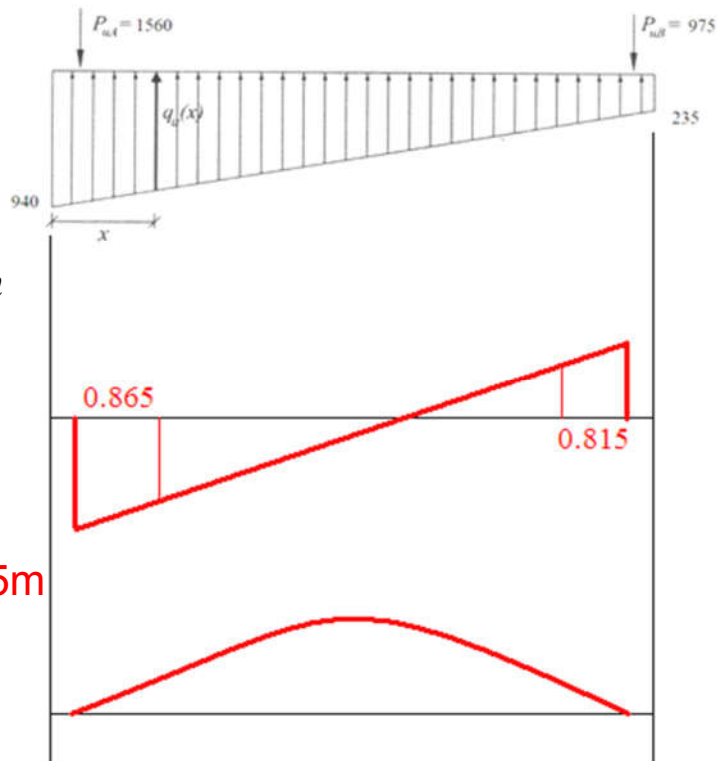
Convert trapezoidal load to rectangle

$$w_{ave} = 235 + \frac{2}{3}(940 - 235) = 705$$

$$-M_{max} = \frac{wl^2}{8} = \frac{705(3.65)^2}{8} = 1174 \text{ kN.m}$$

Clear distance between column

B in moment design = ave. width = 2.5m



Check for beam shear

$$d = 665 \text{ mm}$$

$$b = 1 + 2\left(\frac{x}{L}\right) \times y$$

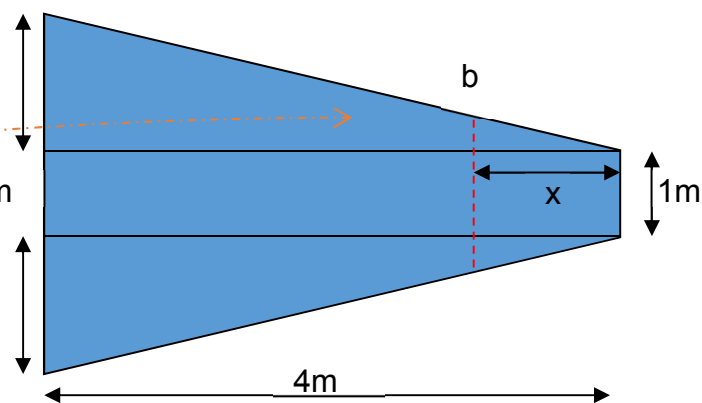
$$\text{at } x = 0.815 + 0.15$$

$$1 + 2\left(\frac{0.965}{4.35}\right) \times 1.5 = 1.7 \text{ m} = 1700 \text{ mm}$$

$$\phi V_C = 0.75 \times \frac{\sqrt{25}}{6} \times 665 \times 1700 / 1000 = 696 \text{ kN}$$

$$\text{Max.} \rightarrow V_U \text{ at } \underline{d} \text{ from column B face (the most critical)} = 668 \text{ kN}$$

$$V_U < \phi V_C$$



Bending moment Long direction

$$b = 1 + 2\left(\frac{2.25}{4.35}\right) \times 1.5 = 2.60m = 2600 \quad \text{Top}$$

$$-ve M = 1260kN.m$$

$$d = 730mm$$

$$\rho = 0.85 * \frac{25}{420} \left[1 - \sqrt{1 - \frac{2 \times 10^6 * 1260}{0.9(0.85)25 * 665^2 * 2600}} \right] = 0.003$$

$$A_s = 0.003 \times 665 \times 1000 = 1995mm^2 = 20cm^2 \quad \text{use } 10\phi 16 / m \quad \text{Top}$$

Bottom

$$A_{s_{min}} = 0.0018 \times 750 \times 1000 = 1350mm^2 = 13.5cm^2 \quad \text{use } 9\phi 14 / m \quad \text{Bottom}$$

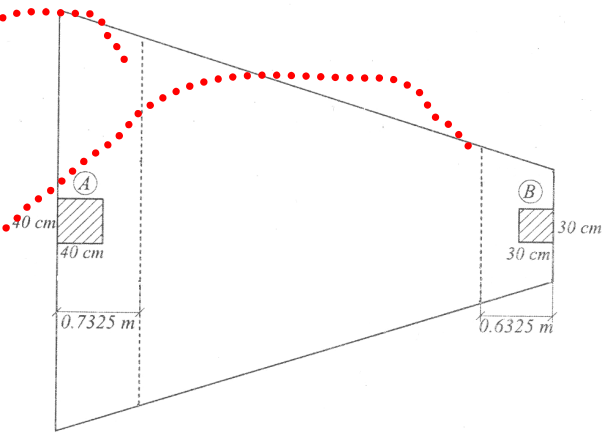
Bending moment Short direction

Under Column A

$$b' = 1 + 2\left(\frac{3.62}{4.35}\right) \times 1.5 = 3.5m = 3500mm$$

$$b = \frac{3.5 + 4}{2} m = 3.75m = 3750mm$$

$$b' = 1 + 2\left(\frac{0.633}{4.35}\right) \times 1.5 = 1.44m = 1440mm$$



$$M = \frac{1560}{(3.75 * 0.733)} \times \frac{0.733}{2} \left(\frac{3.75 - 0.4}{2} \right)^2 = 583.6$$

$$d = 665mm$$

$$\rho = 0.85 * \frac{25}{420} \left[1 - \sqrt{1 - \frac{2 \times 10^6 * 583.57}{0.9(0.85)25 * 665^2 * 733}} \right] = 0.005$$

$$A_s = 0.005 \times 665 \times 733 = 3325mm^2 = 33cm^2 \quad \text{use } 10\phi 20$$

Under Column B

$$b = \frac{1.44 + 1}{2} m = 1.22 m = 1220 mm \quad \rho = 0.85 \times \frac{f_c}{f_y} \left[1 - \sqrt{1 - \frac{M}{0.9 \times 0.86 \times b \times d}} \right]$$

$$M = \frac{975}{(1.22 \times 0.633)} \times \frac{0.633}{2} \left(\frac{1.22 - 0.3}{2} \right)^2 = 84.6$$

$$b = 633 mm, d = 665 mm$$

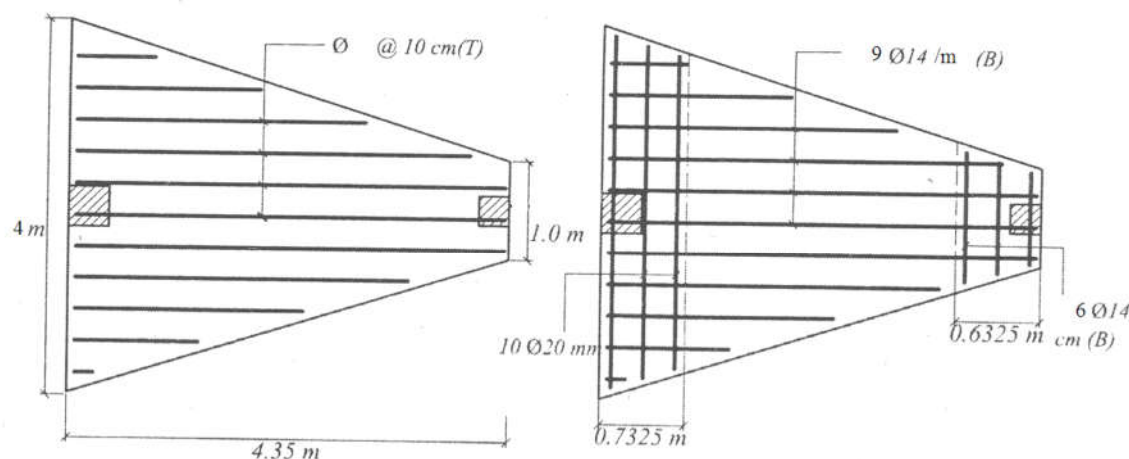
$$\rho = 0.85 \times \frac{25}{420} \left[1 - \sqrt{1 - \frac{2 \times 10^6 \times 84.6}{0.9 (0.85) 25 \times 665^2 \times 633}} \right] < \rho_{\min}$$

$$A_{S_{\min}} = 0.0018 \times 750 \times 633 = 854.6 mm^2 = 8.6 cm^2 \quad \text{use } 6\phi 14$$

Shrinkage Reinforcement in short direction

$$A_{S_{\min}} = 0.0018 \times 1000 \times 750 = 1350 mm^2 = 13.5 cm^2 \quad \text{use } 9\phi 14 / m$$

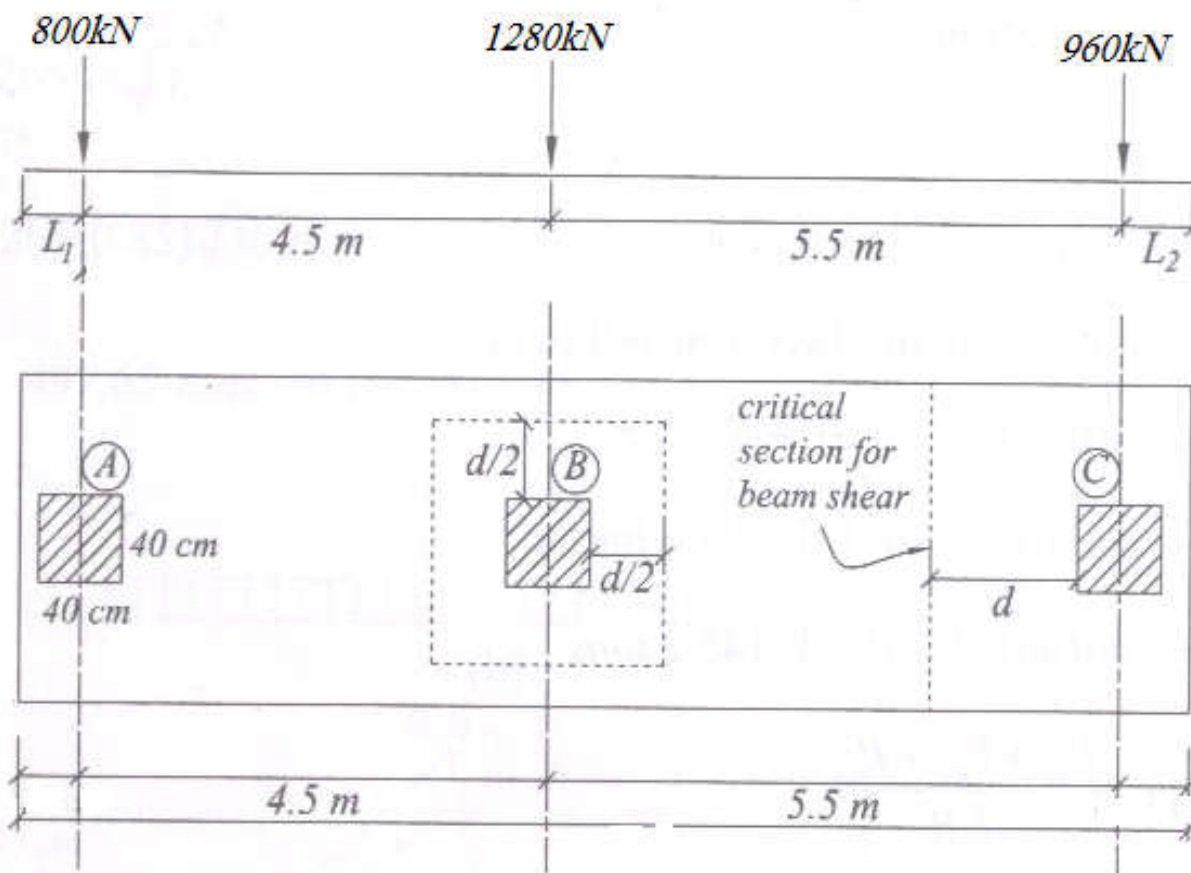
Reinforcement details



Example 3 (Strip footing)

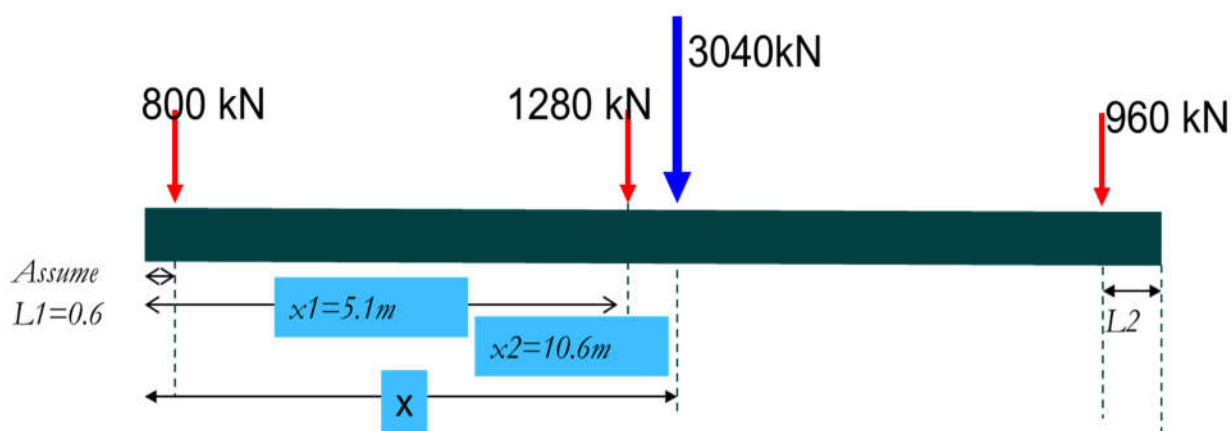
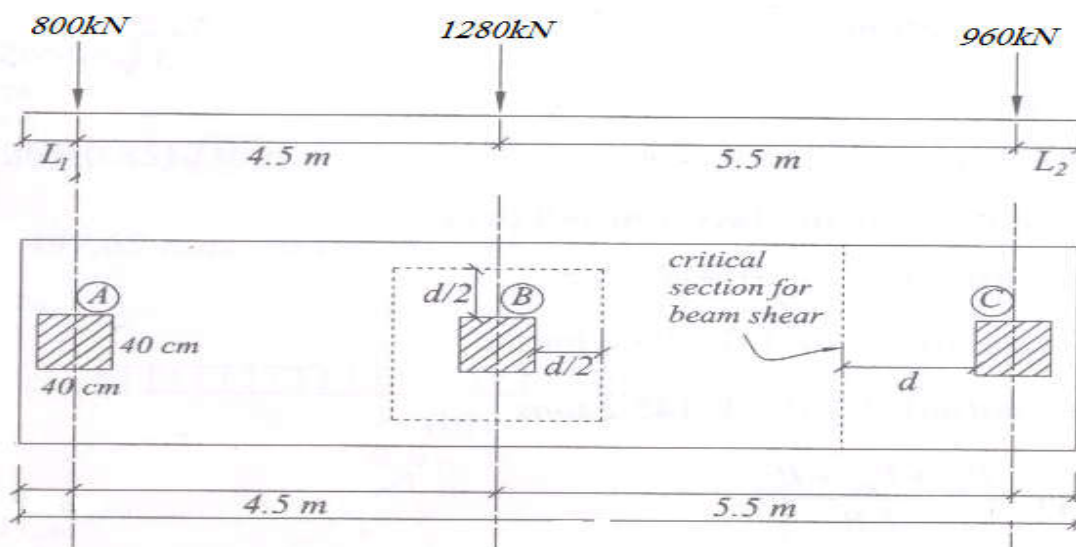
Design a combined or continue footing as shown

$$q_{all(net)} = 20t / m^2 = 200kPa \quad f'_c = 25 N/mm^2 \quad f_y = 420 N/mm^2$$



Dimension calculation

The base dimension to get uniform distributed load



$$800(0.6) + 1280(5.1) + 960(10.6) = 3040(x)$$

$$x = 5.65m,$$

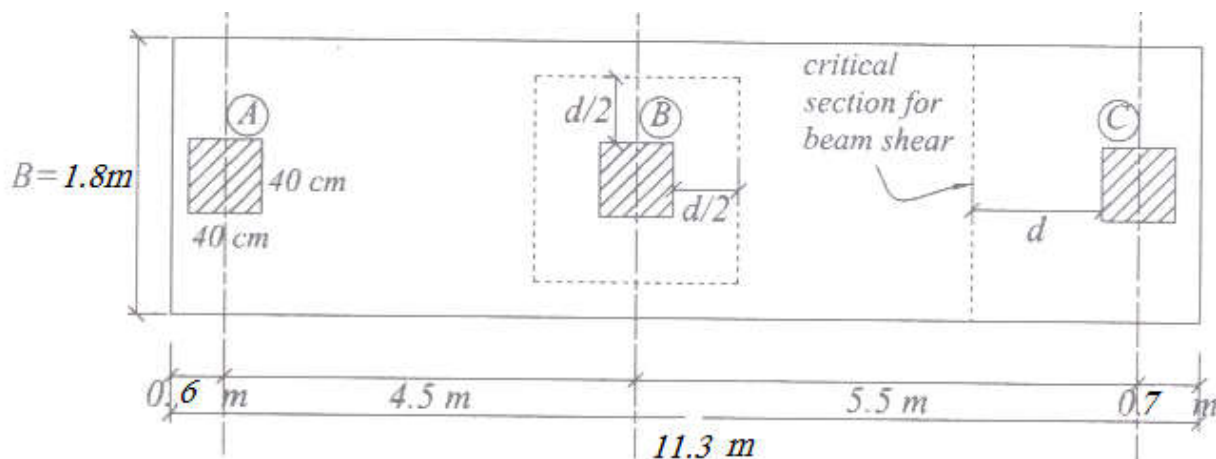
$$2(x) = 11.3m$$

$$L2 = 11.3 - (10.6) = 0.7$$

$$q_{all(net)} = 18t / m^2 = 180kPa,$$

$$A_g = \frac{P_s}{q_{all(net)}} = \frac{3040 \times 10^3}{180 \times 10^3} = 16.9m^2 \approx 11.3m \times 1.8m$$

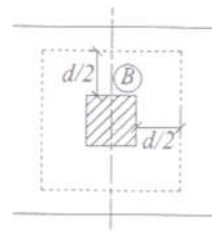
$$q_u = \frac{P_u}{A} = \frac{1.3(3040) \times 10^3}{11.3 \times 1.8} = 195 \times 10^3 Pa = 195kPa$$



Check for punching Shear

$$h = 700 \text{ mm}$$

$$d = 630 \text{ mm}$$



Column B

$$b_o = 4(630 + 400) = 4120 \text{ mm}$$

$$\phi V_c = \phi \frac{\sqrt{f'_c}}{3} b_o d = 0.75 \times \frac{\sqrt{25}}{3} \times 630 \times 4120 / 1000 = 3244.5 \text{ kN}$$

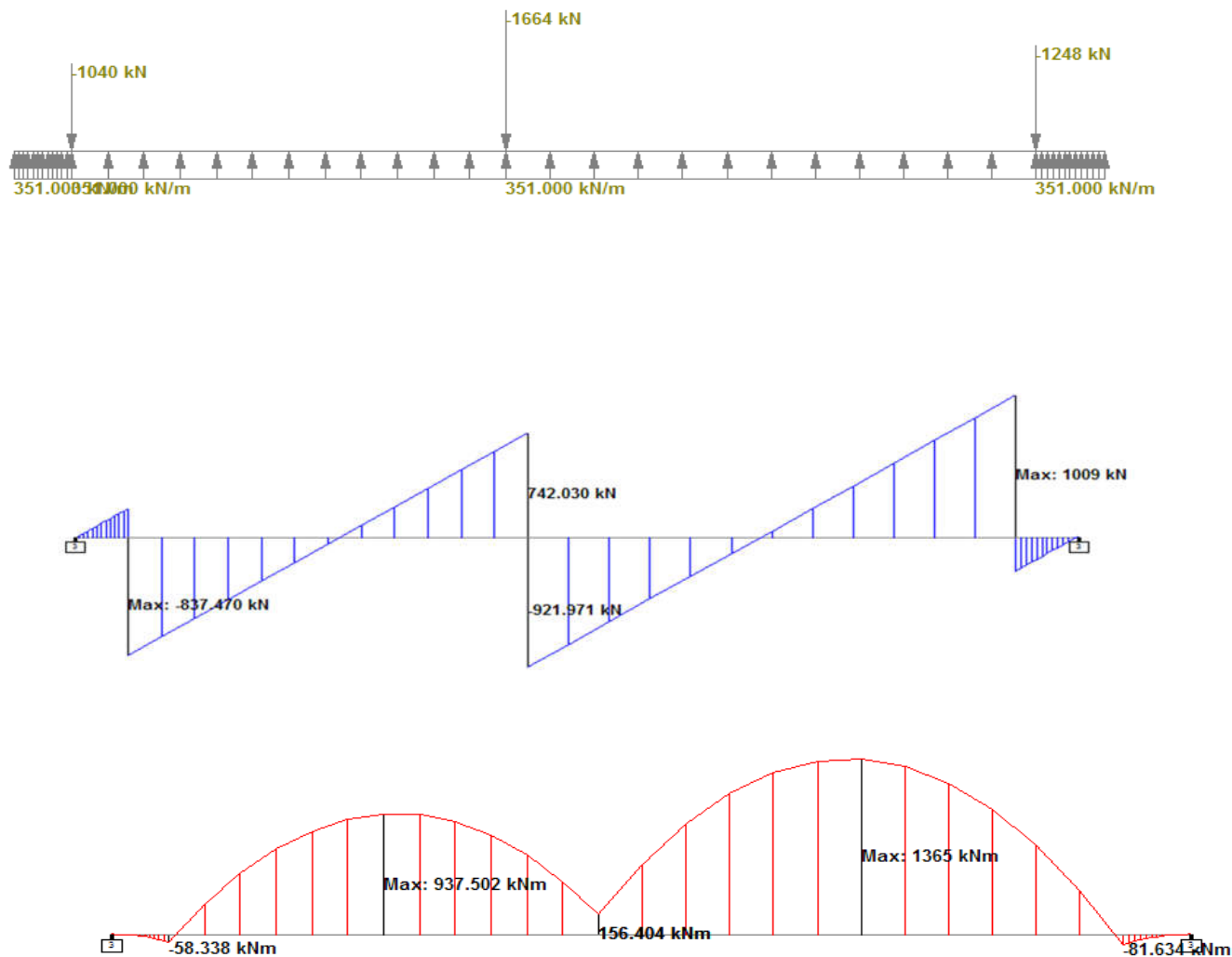
$$\phi V_c = \phi \left(2 + \frac{\alpha_s d}{b} \right) \frac{\sqrt{f'_c}}{12} b_o d = 0.75 \left(2 + \frac{40 \times 630}{4120} \right) \times \frac{\sqrt{25}}{12} \times 630 \times 4120 / 1000 = 6584 \text{ kN}$$

$$V_u = 1280(1.3) - 1.03^2 * 195 = 1457.1 \text{ kN} < \phi V_c \quad \text{oK}$$

You can check other columns

Draw S.F.D & B.M.D

$$\text{Stress under footing} \\ = 195 * 1.8 = 351 \text{ kN/m}$$



Check for beam shear

$$b = 1800\text{mm}, d = 630\text{mm}$$

$$\phi V_C = 0.75 \times \frac{\sqrt{25}}{6} \times 630 \times 1800 / 1000 = 708.75\text{kN}$$

$$\text{Max} \rightarrow V_U \text{ at } \underline{d} \text{ from column face} \approx 0.7(1009) = 706.3\text{kN}$$

$$V_U < \phi V_C$$

Bending moment Long direction

$$-ve M = 1366\text{kN.m}$$

$$b = 1800\text{mm}, d = 730\text{mm}$$

$$\rho = 0.85 \times \frac{f_c}{f_y} \left[1 - \sqrt{1 - \frac{M}{0.9 \times 0.86 \times b \times d}} \right]$$

$$\rho = 0.85 * \frac{25}{420} \left[1 - \sqrt{1 - \frac{2 \times 10^6 * 1365}{0.9(0.85)25 * 630^2 * 1800}} \right] = 0.0053$$

$$A_S = 0.0053 \times 630 \times 1000 = 3362\text{mm}^2 = 33.6\text{cm}^2 \text{ use } 9\phi 22 / m \text{ Top}$$

$$+ve M = 246.7\text{kN.m}$$

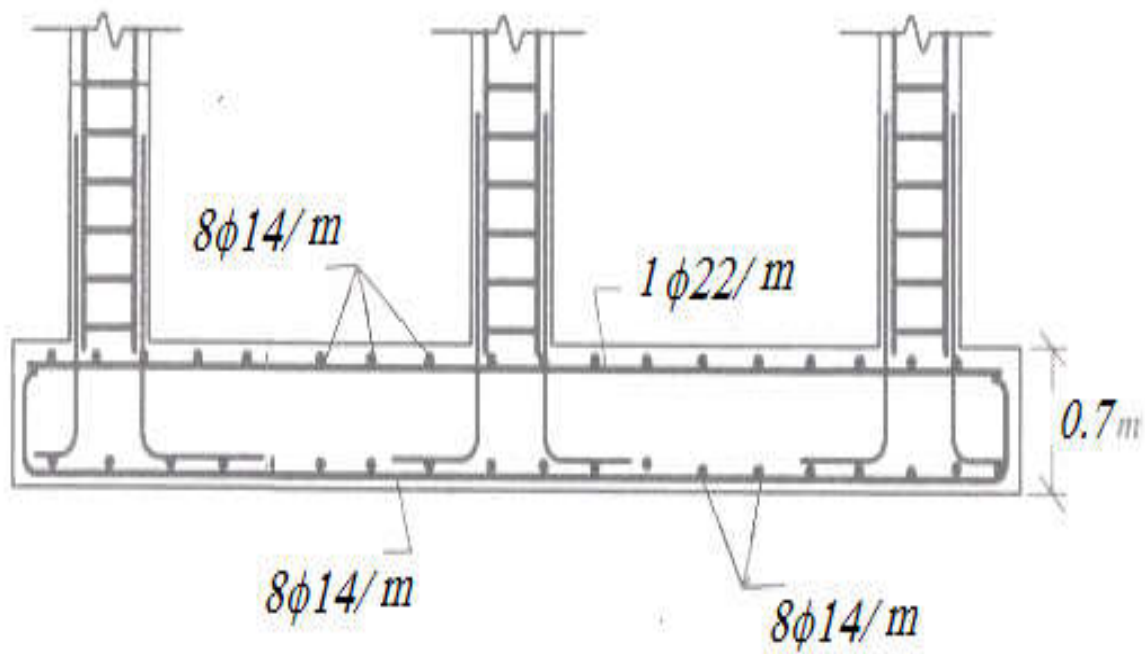
$$b = 1800\text{mm}, d = 730\text{mm}$$

$$\rho = 0.85 * \frac{25}{420} \left[1 - \sqrt{1 - \frac{2 \times 10^6 * 81}{0.9(0.85)25 * 730^2 * 1800}} \right] < \rho_{\min}$$

$$A_{S_{\min}} = 0.0018 \times 700 \times 1000 = 1260\text{mm}^2 = 12.6\text{cm}^2 \text{ use } 8\phi 14 / m \text{ Bottom}$$

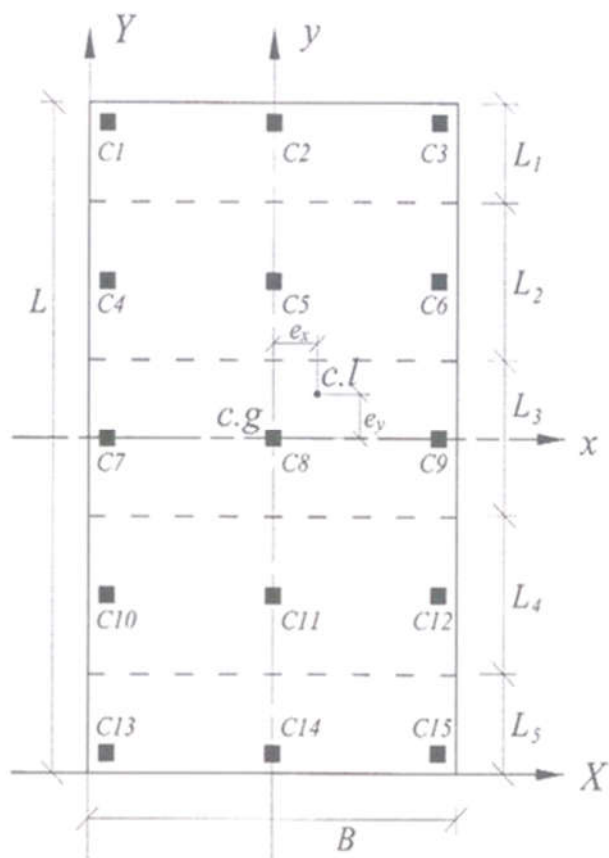
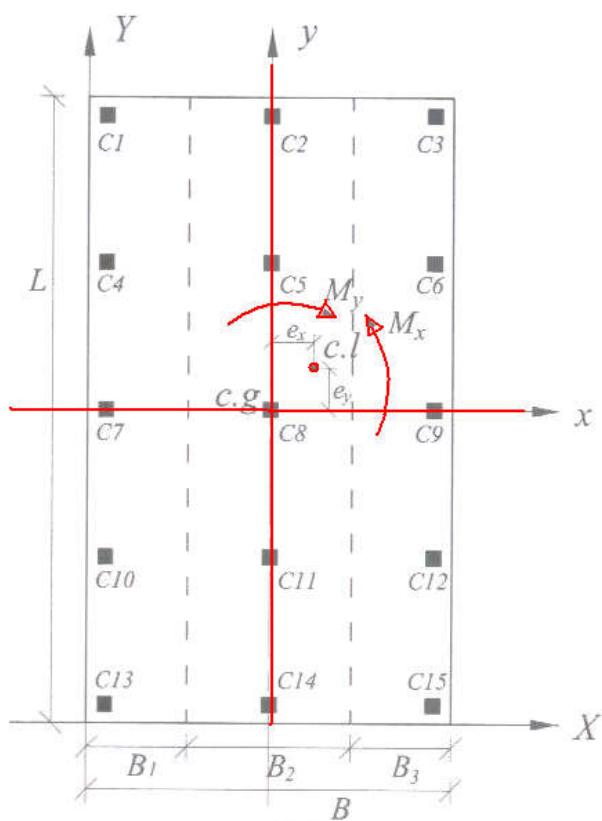
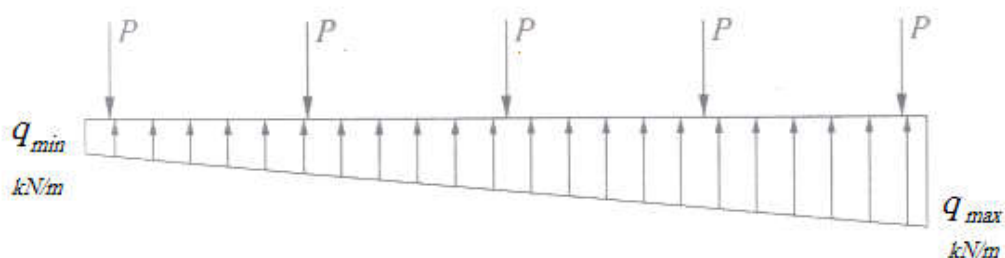
Design Short direction as example 1 (lecture 11)

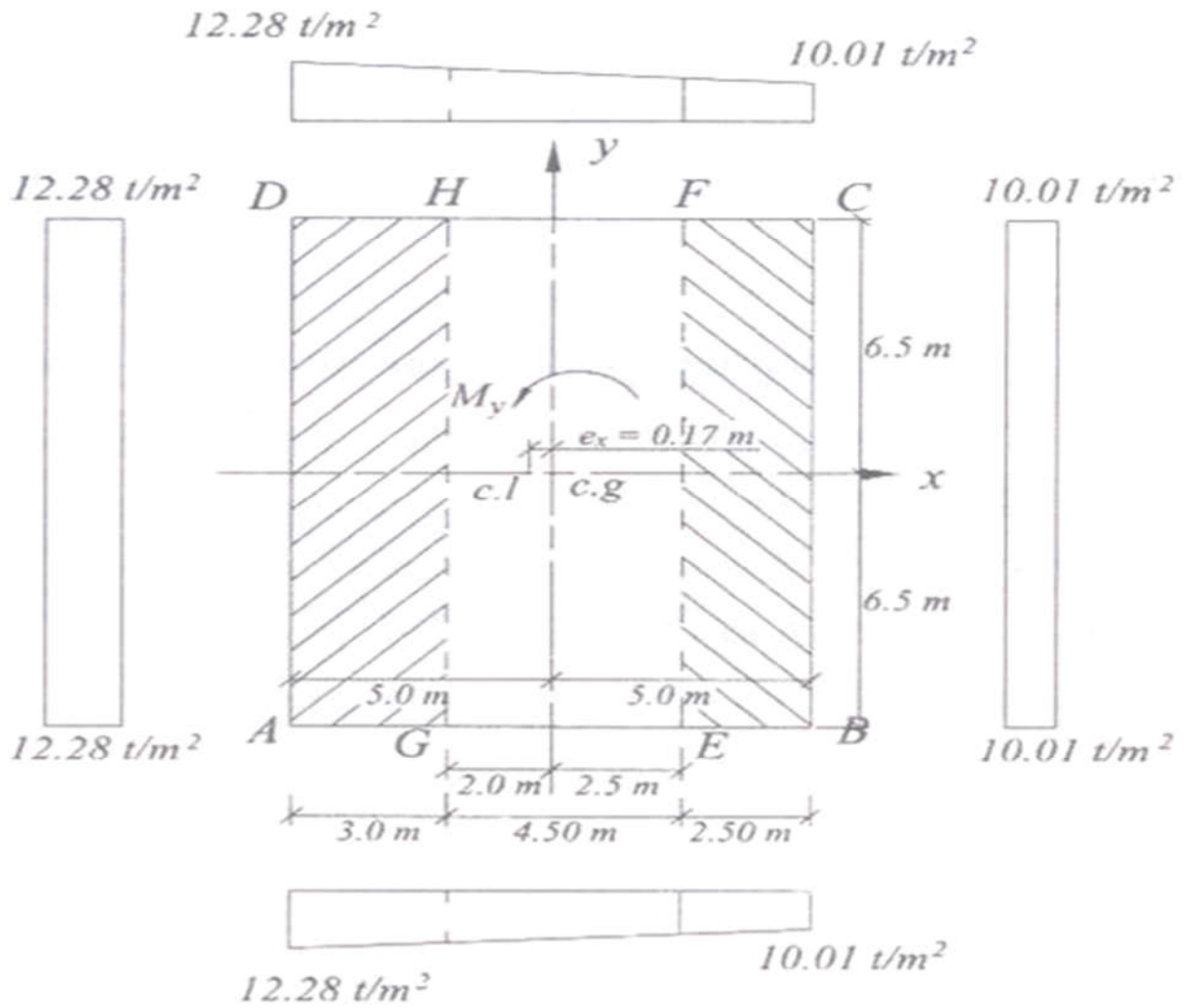
Reinforcement details



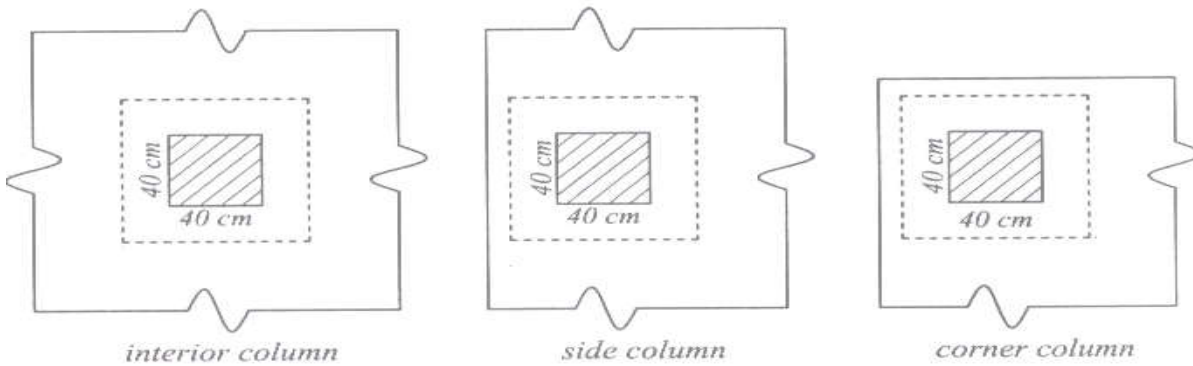
Mat Foundation

$$q = \frac{\sum P}{A} \pm \frac{M_x y}{I_x} \pm \frac{M_y x}{I_y}$$



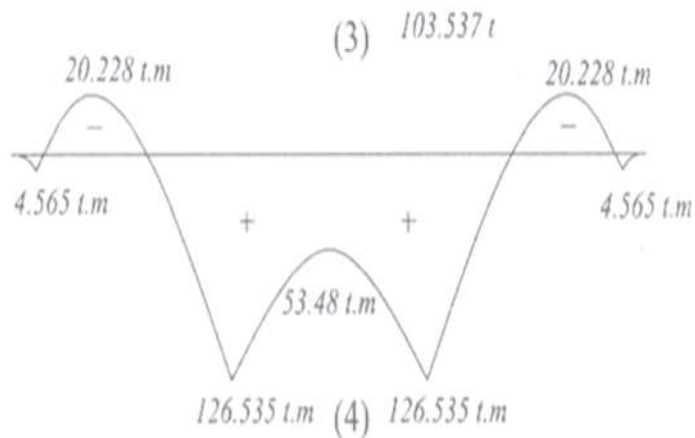
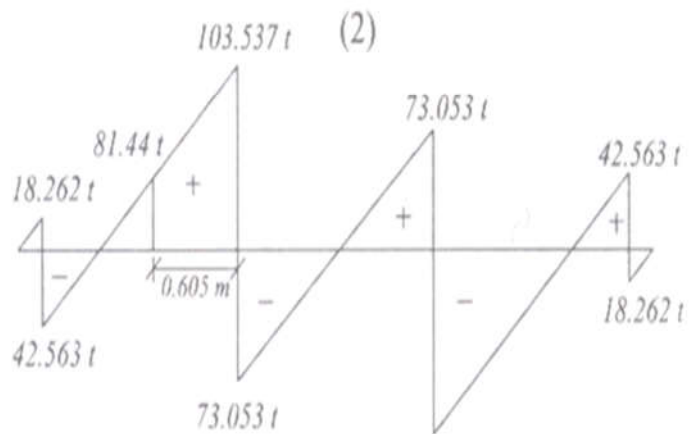
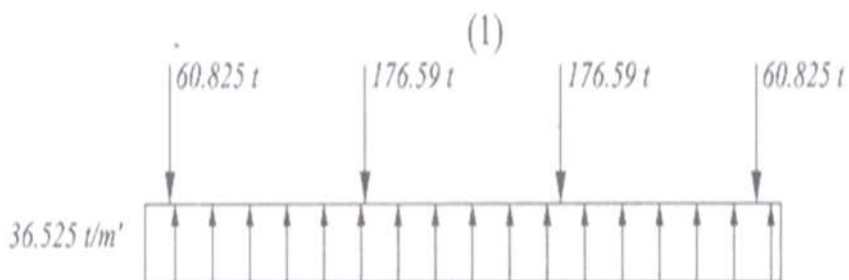
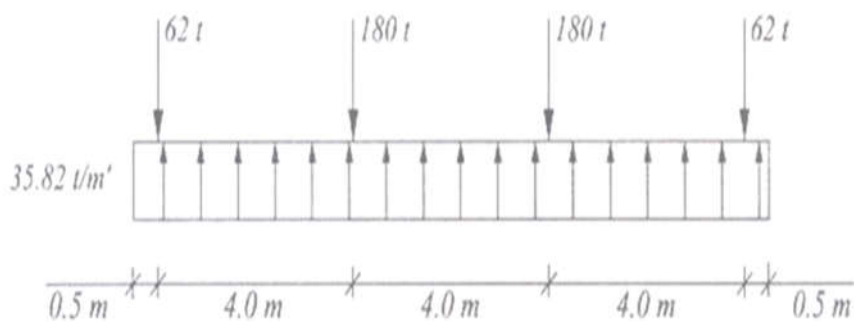


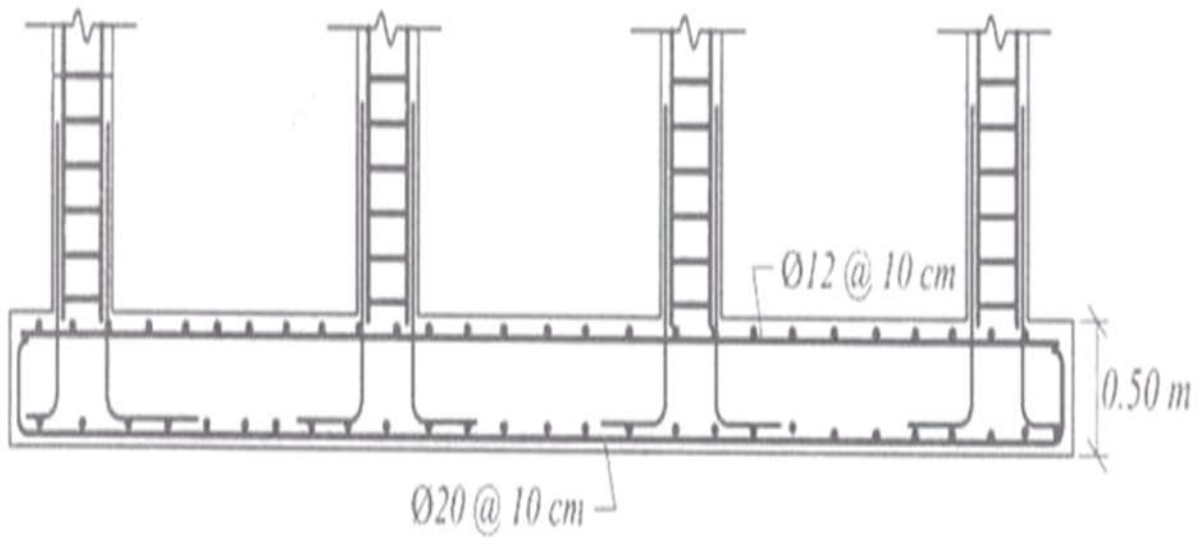
Check for punching Shear



General Example, ^{Ref. 3}

Modified load





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