BANGLADESH UNIVERSITY OF PROFESSIONALS
Military Institute of Science and Technology
B.Sc. in Environmental, Water Resources, and Coastal Engineering, Term Final (Spring) Examination 2025: Oct - Nov 2025

Student Group: 72<Earned Credit Hours ≤ 108

Subject: CE 385, Design of Concrete Structures-I

Time: 3.00 hours **Full Marks: 180**

INSTRUCTIONS:

a. Use **SEPARATE** answer scripts for each section.
b. **Question-1** in **Section-A** and **Question-5** in **Section-B** are compulsory.
c. Answer any other **TWO** out of the remaining **THREE** questions from each section.
d. Figures in the margin indicate full **marks**.
e. Assume reasonable data if necessary.
f. **Symbols** and **abbreviations** used have their usual meanings.
g. **Table and charts are attached at Annex**

SECTION-A

Question 1 **(Compulsory)**

A reinforced concrete slab is built integrally with its supports and consists of three equal spans, each with a clear span of 15 ft. The service live load is 100 psf, and 4000 psi concrete is specified for use with steel with a yield stress equal to 60,000 psi. Design the slab, following the provisions of the ACI code. **30**

Question 2

a. Derive the following equation for the balanced ratio of a reinforced concrete beam (provide diagrams where necessary) **20**

$$\rho_b = 0.85\beta_1 \frac{f_c'}{f_y} \times \frac{c}{d} = 0.85\beta_1 \frac{f_c'}{f_y} \times \frac{\epsilon_u}{\epsilon_u + \epsilon_y}$$

Notations follow their usual meanings.

b. Explain why the upper limit of reinforcement ratio should be below the balanced reinforcement ratio in actual practice of designing beam. **10**

Question 3

a. A rectangular beam that must carry a service live load of 2.5 kips/ft and a calculated dead load of 1.1 kips/ft on an 18 ft simple span is limited in cross section for architectural reasons to 10 inch width and 20 inch total depth. If f_y = 60,000 psi and f'_c = 4000 psi, what steel area(s) must be provided? **30**

Question 4

a. Design a slab panel 20 ft by 25 ft (clear span) to support a uniform live load of 100 psf. The panel is a corner panel of a group. A 3000 psi concrete and steel with f_y = 60,000 psi are used in connection with USD method. Design the slab, following provisions of the ACI code. **30**

SECTION-B

Question 5 (Compulsory)

a. Design a singly reinforced beam for flexure, the dead load of the beam is 2.5kip/ft and live load is 2 kip/ft on a 25 feet span. The width of the beam is limited to 16 inch. Also show the reinforcement detailing of beam section with neat sketches. Given, $f'_c = 4$ ksi and $f_y = 60$ ksi. **20**

b. Briefly describe the locations of critical sections considered for shear design. **5**

c. Briefly describe any two methods by which web reinforcement enhances the shear resistance of a reinforced concrete beam after diagonal cracks develop. **5**

Question 6

a. Compare the cracking moment based on the gross section properties for the beam shown in Figure-6(a). Given, yield point of steel, $f_y = 72{,}500$ psi and concrete compressive strength, $f'_c = 5000$ psi. Tensile capacity of concrete is $7.5\sqrt{f'_c}$ (psi). **10**

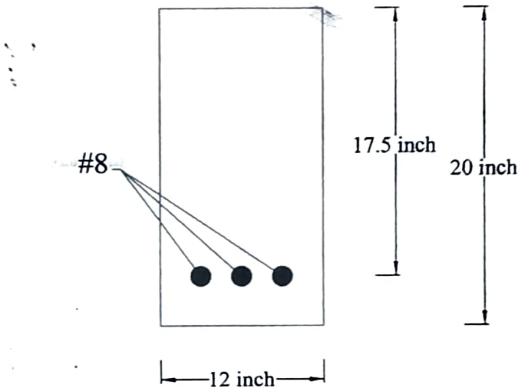


Figure-6(a)

b. A simply supported rectangular beam and loads are shown in Figure-6(b). The beam carries a total factored load of 8 kips/ft on a 20 feet clear span. Design the beam for shear. Show the detailing of the stirrups in a neat sketch. Given, $f_y = 60{,}000$ psi and $f'_c = 4000$ psi for normal weight concrete. **20**

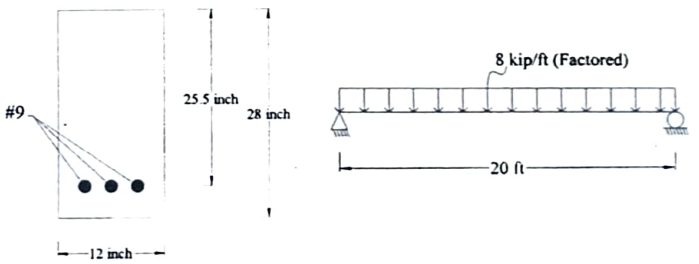


Figure -6(b)

Question 7

a. Briefly describe the types of bond failure for reinforcing bars in tension. **5**

b. A beam-column joint in a continuous building frame is shown in Figure-7(b). Based on the frame analysis, the negative steel required at the end of the beam is 2.58 in^2. Two #11 bars are used. Beam dimensions are b = 10 inch, d = 18 inch, and h = 21 inch. The design **15**

will include #3 stirrups spaced four at 4 inch, followed by a constant 6 inch spacing in the region of the support with 1.5 inch clear cover. Normal weight concrete is to be used, with f'_c= 4 ksi and f_y= 60 ksi. Find the minimum distance l_d, at which the negative bars can be cut off, based on development of the required steel area at the face of the column using the more accurate equation for deformed bars or deformed wires according to ACI code 25.4.2.3.

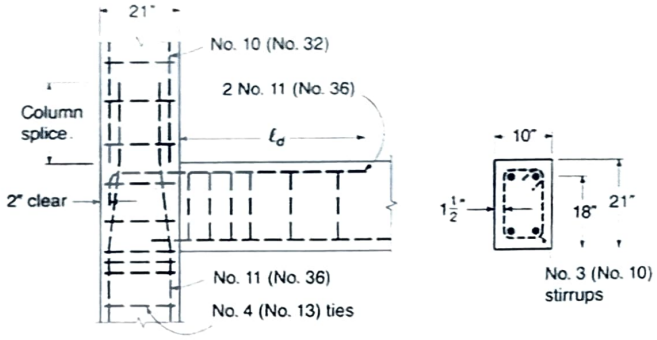


Figure-7(b)

c. Referring to the beam-column joint, shown in Figure-7(b), the #11 negative bars are to be extended into the column and terminated in a standard 90° hook, keeping 2 inch clear to the outside face of the column. The column width in the direction of beam width is 16 inch. Find the minimum length of embedment of the hook past the column face, and specify the hook details. **10**

Question 8

a. State the factors influencing development length of reinforcing bars in tension. **5**

b. Three #6 bars serve as top reinforcement for a bracket framing into a 15×15 inch column, shown in Figure-8(b). The bracket projects 14 inch from the column and is the same width as the column. The top cover to the center of the bars is 3.5 inch. The bars are spaced laterally at 4.5 inch. These dimensions are inadequate for straight development length or for hooked bars with standard hooks. Based on other reinforcement, cover requirements, and head thickness, total development lengths for headed bars of 12.5 inch in the column and 11.5 inch in the bracket are available. The reinforcing bars have f_y= 60,000 psi and the concrete is normal-weight concrete with f'_c= 4500 psi. Determine if a bar with heads at both ends can be used in this application. **10**

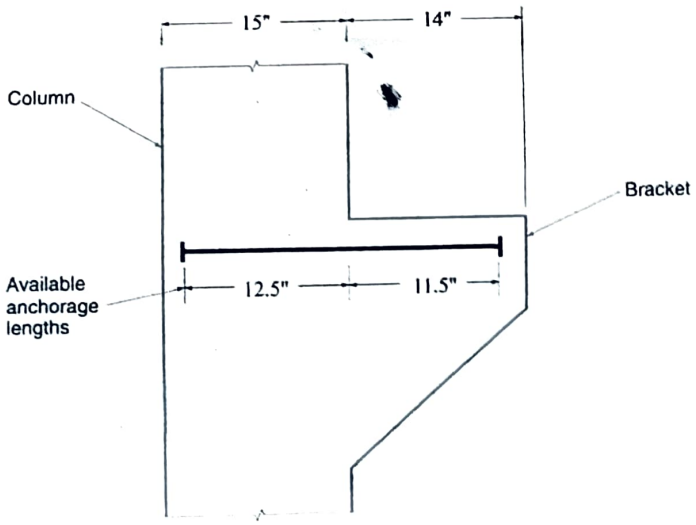


Figure-8(b)

c. A simply –supported rectangular beam shown in Figure-8(c) carries a total factored uniformly distributed load of 4 kip/ft on a span of 30 feet. Determine the design moment of the beam and check if it is adequate to carry the load.

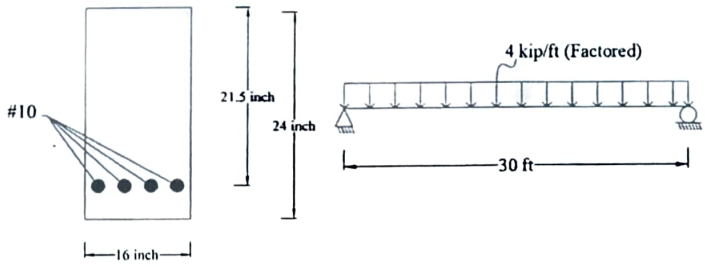


Figure-8(c)

Annexure

TABLE 12.1
Minimum thickness *h* of nonprestressed one-way slabs

Simply supported	$\ell/20$
One end continuous	$\ell/24$
Both ends continuous	$\ell/28$
Cantilever	$\ell/10$

Table A2.1 ACI approximate design moments and shears for beams and one-way slabs

Positive moment	End spans	
	Discontinuous end unrestrained	$w_u\ell_n^2/11$
	Discontinuous end integral with support	$w_u\ell_n^2/14$
	Interior spans	$w_u\ell_n^2/16$
Negative moment	At exterior face of the first interior support	
	Two spans	$w_u\ell_n^2/9$
	More than two spans	$w_u\ell_n^2/10$
	At other faces of interior supports	$w_u\ell_n^2/11$
	At the face of all supports for SLABS with spans not exceeding 10 ft; and BEAMS where ratio of sum of column stiffnesses to beam stiffness exceeds 8 at each end of the span	$w_u\ell_n^2/12$
	At interior face of exterior support for members built integrally with supports	
	Where support is a spandrel beam	$w_u\ell_n^2/24$
	Where support is a column	$w_u\ell_n^2/16$
Shear	In end members at the face of the first interior support	$1.15\,w_u\ell_n/2$
	At face of all other supports	$w_u\ell_n/2$

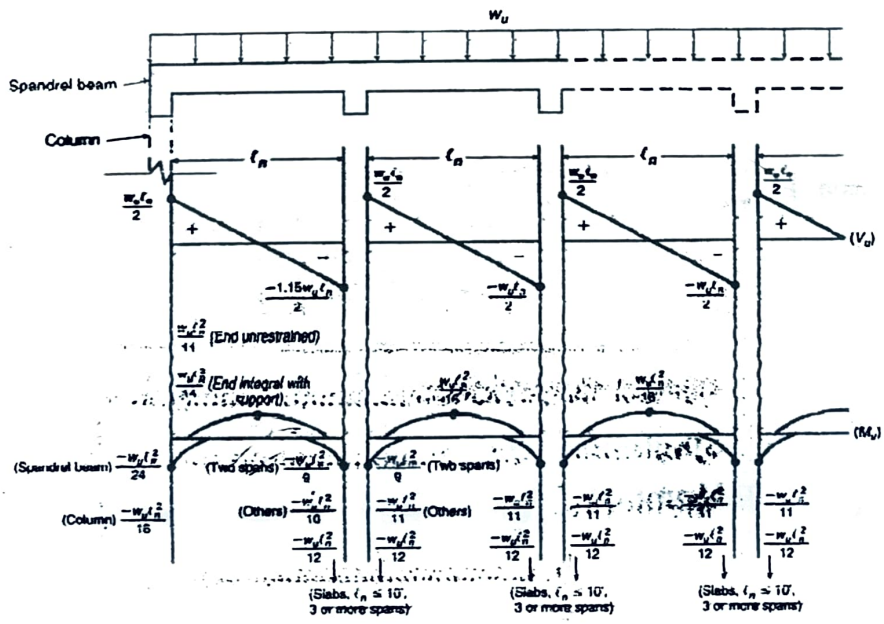


Table 1 - Coefficients for Negative Moments in Slabs

$M_a^- = C_{a,neg} w_u l_a^2$

$M_b^- = C_{b,neg} w_u l_b^2$

where w_u = total factored uniform load (DL + LL)

Ratio $m = \frac{l_a}{l_b}$		Case 1	Case 2	Case 3	Case 4	Case 5	Case 6	Case 7	Case 8	Case 9
1.00	$C_{a,neg}$	-	0.045	0.076	0.050	0.075	0.071	0.071	0.033	0.061
	$C_{b,neg}$		0.045		0.050				0.061	0.033
0.95	$C_{a,neg}$	-	0.050	0.072	0.055	0.079	0.075	0.067	0.038	0.065
	$C_{b,neg}$		0.041		0.045				0.056	0.029
0.90	$C_{a,neg}$	-	0.055	0.070	0.060	0.080	0.079	0.062	0.043	0.068
	$C_{b,neg}$		0.037		0.040				0.052	0.025
0.85	$C_{a,neg}$	-	0.060	0.065	0.066	0.082	0.083	0.057	0.049	0.072
	$C_{b,neg}$		0.031		0.034				0.046	0.021
0.80	$C_{a,neg}$	-	0.065	0.061	0.071	0.083	0.086	0.051	0.055	0.075
	$C_{b,neg}$		0.027		0.029				0.041	0.017
0.75	$C_{a,neg}$	-	0.069	0.056	0.076	0.085	0.088	0.044	0.061	0.078
	$C_{b,neg}$		0.022		0.024				0.036	0.014
0.70	$C_{a,neg}$	-	0.074	0.050	0.081	0.086	0.091	0.038	0.068	0.081
	$C_{b,neg}$		0.017		0.019				0.029	0.011
0.65	$C_{a,neg}$	-	0.077	0.043	0.085	0.087	0.093	0.031	0.074	0.083
	$C_{b,neg}$		0.014		0.015				0.024	0.008
0.60	$C_{a,neg}$	-	0.081	0.035	0.089	0.088	0.095	0.024	0.080	0.085
	$C_{b,neg}$		0.010		0.011				0.018	0.006
0.55	$C_{a,neg}$	-	0.084	0.028	0.092	0.089	0.096	0.019	0.085	0.086
	$C_{b,neg}$		0.007		0.008				0.014	0.005
0.50	$C_{a,neg}$	-	0.086	0.022	0.094	0.090	0.097	0.014	0.089	0.088
	$C_{b,neg}$		0.006		0.006				0.010	0.003

Table 2 - Coefficients for Dead Load Positive Moments in Slabs

$M_{a,DL}^{+} = C_{a,DL} w_{DL} l_a^2$

$M_{b,DL}^{+} = C_{b,DL} w_{DL} l_b^2$

where w_{DL} = uniform factored Dead Load (DL)

Ratio $m = \frac{l_a}{l_b}$		Case 1	Case 2	Case 3	Case 4	Case 5	Case 6	Case 7	Case 8	Case 9
1.00	$C_{a,DL}$	0.036	0.018	0.018	0.027	0.027	0.033	0.027	0.020	0.023
	$C_{b,DL}$	0.036	0.018	0.027	0.027	0.018	0.027	0.033	0.023	0.020
0.95	$C_{a,DL}$	0.040	0.020	0.021	0.030	0.028	0.036	0.031	0.022	0.024
	$C_{b,DL}$	0.033	0.016	0.025	0.024	0.015	0.024	0.031	0.021	0.017
0.90	$C_{a,DL}$	0.045	0.022	0.025	0.033	0.029	0.039	0.035	0.025	0.026
	$C_{b,DL}$	0.029	0.014	0.024	0.022	0.013	0.021	0.028	0.019	0.015
0.85	$C_{a,DL}$	0.050	0.024	0.029	0.036	0.031	0.042	0.040	0.029	0.028
	$C_{b,DL}$	0.026	0.012	0.022	0.019	0.011	0.017	0.025	0.017	0.013
0.80	$C_{a,DL}$	0.056	0.026	0.034	0.039	0.032	0.045	0.045	0.032	0.029
	$C_{b,DL}$	0.023	0.011	0.020	0.016	0.009	0.015	0.022	0.015	0.010
0.75	$C_{a,DL}$	0.061	0.028	0.040	0.043	0.033	0.048	0.051	0.036	0.031
	$C_{b,DL}$	0.019	0.009	0.018	0.013	0.007	0.012	0.020	0.013	0.007
0.70	$C_{a,DL}$	0.068	0.030	0.046	0.046	0.035	0.051	0.058	0.040	0.033
	$C_{b,DL}$	0.016	0.007	0.016	0.011	0.005	0.009	0.017	0.011	0.006
0.65	$C_{a,DL}$	0.074	0.032	0.054	0.050	0.036	0.054	0.065	0.044	0.034
	$C_{b,DL}$	0.013	0.006	0.014	0.009	0.004	0.007	0.014	0.009	0.005
0.60	$C_{a,DL}$	0.081	0.034	0.062	0.053	0.037	0.056	0.073	0.048	0.036
	$C_{b,DL}$	0.010	0.004	0.011	0.007	0.003	0.006	0.012	0.007	0.004
0.55	$C_{a,DL}$	0.088	0.035	0.071	0.056	0.038	0.058	0.081	0.052	0.037
	$C_{b,DL}$	0.008	0.003	0.009	0.005	0.002	0.004	0.009	0.005	0.003
0.50	$C_{a,DL}$	0.095	0.037	0.080	0.059	0.039	0.061	0.089	0.056	0.038
	$C_{b,DL}$	0.006	0.002	0.007	0.004	0.001	0.003	0.007	0.004	0.002

Table 3 - Coefficients for Live Load Positive Moments in Slabs

$M_{a,LL}^{+} = C_{a,LL} w_{LL} l_a^2$

$M_{b,LL}^{+} = C_{b,LL} w_{LL} l_b^2$

where w_{LL} = uniform factored Live Load (LL)

Ratio $m = \frac{l_a}{l_b}$		Case 1	Case 2	Case 3	Case 4	Case 5	Case 6	Case 7	Case 8	Case 9
1.00	$C_{a,LL}$	0.036	0.027	0.027	0.032	0.032	0.035	0.032	0.028	0.030
	$C_{b,LL}$	0.036	0.027	0.032	0.032	0.027	0.032	0.035	0.030	0.028
0.95	$C_{a,LL}$	0.040	0.030	0.031	0.035	0.034	0.038	0.036	0.031	0.032
	$C_{b,LL}$	0.033	0.025	0.029	0.029	0.024	0.029	0.032	0.027	0.025
0.90	$C_{a,LL}$	0.045	0.034	0.035	0.039	0.037	0.042	0.040	0.035	0.036
	$C_{b,LL}$	0.029	0.022	0.027	0.026	0.021	0.025	0.029	0.024	0.022
0.85	$C_{a,LL}$	0.050	0.037	0.040	0.043	0.041	0.046	0.045	0.040	0.039
	$C_{b,LL}$	0.026	0.019	0.024	0.023	0.019	0.022	0.026	0.022	0.020
0.80	$C_{a,LL}$	0.056	0.041	0.045	0.048	0.044	0.051	0.051	0.044	0.042
	$C_{b,LL}$	0.023	0.017	0.022	0.020	0.016	0.019	0.023	0.019	0.017
0.75	$C_{a,LL}$	0.061	0.045	0.051	0.052	0.047	0.055	0.056	0.049	0.046
	$C_{b,LL}$	0.019	0.014	0.019	0.016	0.013	0.016	0.020	0.016	0.013
0.70	$C_{a,LL}$	0.068	0.049	0.057	0.057	0.051	0.060	0.063	0.054	0.050
	$C_{b,LL}$	0.016	0.012	0.016	0.014	0.011	0.013	0.017	0.014	0.011
0.65	$C_{a,LL}$	0.074	0.053	0.064	0.062	0.055	0.064	0.070	0.059	0.054
	$C_{b,LL}$	0.013	0.010	0.014	0.011	0.009	0.010	0.014	0.011	0.009
0.60	$C_{a,LL}$	0.081	0.058	0.071	0.067	0.059	0.068	0.077	0.065	0.059
	$C_{b,LL}$	0.010	0.007	0.011	0.009	0.007	0.008	0.011	0.009	0.007
0.55	$C_{a,LL}$	0.088	0.062	0.080	0.072	0.063	0.073	0.085	0.070	0.063
	$C_{b,LL}$	0.008	0.006	0.009	0.007	0.005	0.006	0.009	0.007	0.006
0.50	$C_{a,LL}$	0.095	0.066	0.088	0.077	0.067	0.078	0.092	0.076	0.067
	$C_{b,LL}$	0.006	0.004	0.007	0.005	0.004	0.005	0.007	0.005	0.004

Table 4 - **Coefficients for Shear in Slabs**

Ratio of load W in l_a and l_b directions for Shear in Slab and Load on Supports for Beams

Ratio $m = \frac{l_a}{l_b}$		Case 1	Case 2	Case 3	Case 4	Case 5	Case 6	Case 7	Case 8	Case 9
										
1.00	W_a	0.50	0.50	0.17	0.50	0.83	0.71	0.29	0.33	0.67
	W_b	0.50	0.50	0.83	0.50	0.17	0.29	0.71	0.67	0.33
0.95	W_a	0.55	0.55	0.20	0.55	0.86	0.75	0.33	0.38	0.71
	W_b	0.45	0.45	0.80	0.45	0.14	0.25	0.67	0.62	0.29
0.90	W_a	0.60	0.60	0.23	0.60	0.88	0.79	0.38	0.43	0.75
	W_b	0.40	0.40	0.77	0.40	0.12	0.21	0.62	0.57	0.25
0.85	W_a	0.66	0.66	0.28	0.66	0.90	0.83	0.43	0.49	0.79
	W_b	0.34	0.34	0.72	0.34	0.10	0.17	0.57	0.51	0.21
0.80	W_a	0.71	0.71	0.33	0.71	0.92	0.86	0.49	0.55	0.83
	W_b	0.29	0.29	0.67	0.29	0.08	0.14	0.51	0.45	0.17
0.75	W_a	0.76	0.76	0.39	0.76	0.94	0.88	0.56	0.61	0.86
	W_b	0.24	0.24	0.61	0.24	0.06	0.12	0.44	0.39	0.14
0.70	W_a	0.81	0.81	0.45	0.81	0.95	0.91	0.62	0.68	0.89
	W_b	0.19	0.19	0.55	0.19	0.05	0.09	0.38	0.32	0.11
0.65	W_a	0.85	0.85	0.53	0.85	0.96	0.93	0.69	0.74	0.92
	W_b	0.15	0.15	0.47	0.15	0.04	0.07	0.31	0.26	0.08
0.60	W_a	0.89	0.89	0.61	0.89	0.97	0.95	0.76	0.80	0.94
	W_b	0.11	0.11	0.39	0.11	0.03	0.05	0.24	0.20	0.06
0.55	W_a	0.92	0.92	0.69	0.92	0.98	0.96	0.81	0.85	0.95
	W_b	0.08	0.08	0.31	0.08	0.02	0.04	0.19	0.15	0.05
0.50	W_a	0.94	0.94	0.76	0.94	0.99	0.97	0.86	0.89	0.97
	W_b	0.06	0.06	0.24	0.06	0.01	0.03	0.14	0.11	0.03

$$M_n = A_s f_y \left(d - \frac{a}{2} \right) \qquad \rho_{max} = 0.85\beta_1 \frac{f'_c}{f_y} \frac{\varepsilon_u}{\varepsilon_u + 0.005}$$

$$M_n = \rho f_y \left(1 - 0.59 \frac{\rho f_y}{f'_c} \right) bd^2$$

$$a = \frac{A_s f_y}{0.85 f'_c b} \qquad c = a/\beta_1$$

$$\beta_1 = 0.85 - 0.05 \frac{f'_c - 4000}{1000} \text{ and } 0.65 \le \beta_1 \le 0.85$$

$$s = \frac{\phi A_v f_y d}{V_u - \phi V_c} \qquad \rho_{\min} = \frac{3\sqrt{f'_c}}{f_y} \ge \frac{200}{f_y}$$

$$V_c = 2\lambda \sqrt{f'_c} b_w d$$

For $V_s \le 4\sqrt{f'_c} b_w d$,

$$s_{\max} \le \begin{cases} \dfrac{A_v f_{yt}}{0.75\sqrt{f'_c} b_w} \le \dfrac{A_v f_{yt}}{50 b_w} \\ \dfrac{d}{2} \\ 24" \end{cases}$$

Equation for Development Length for Bars and Wires in Tension

According to ACI Code 25.4.2.3, for deformed bars or deformed wires,

$$\ell_d = \left(\frac{3}{40} \frac{f_y}{\lambda \sqrt{f'_c}} \frac{\psi_t \psi_e \psi_s}{\left(\frac{c_b + K_{tr}}{d_b} \right)} \right) d_b \qquad \text{(A)}$$

in which the term $(c_b + K_{tr})/d_b$ may not be taken greater than 2.5. In Eq. (A) terms are defined and values established as follows.

ψ_t = casting position factor
- More than 12 in. of fresh concrete is placed below horizontal reinforcement: 1.3
- Other situations: 1.0

ψ_e = epoxy coating factor
- Epoxy-coated or zinc and epoxy dual-coated bars or wires with cover less than $3d_b$ or clear spacing less than $6d_b$: 1.5
- All other epoxy-coated or zinc and epoxy dual-coated bars or wires: 1.2
- Uncoated and zinc-coated (galvanized) reinforcement: 1.0
- Note: $\psi_t \psi_e$ the product of $\psi_t \psi_e$ need not be taken greater than 1.7.

ψ_s = reinforcement size factor
- No. 6 (No. 19) and smaller bars and deformed wires: 0.8
- No. 7 (No. 22) and larger bars: 1.0

λ = lightweight aggregate concrete factor
- When lightweight aggregate concrete is used: 0.75
- Note: when f_{ct} is specified, $\lambda = f_{ct}/(6.7\sqrt{f_{cm}}) \le 1.0$, where f_{cm} is the measured compressive strength.
- When normalweight concrete is used: 1.0

c_b = spacing or cover dimension, in.
Use the smaller of either the distance from the center of the bar to the nearest concrete surface or one-half the center-to-center spacing of the bars being developed.

K_{tr} = transverse reinforcement index: $40A_{tr}/sn$
- where A_{tr} = total cross-sectional area of all transverse reinforcement that is within the spacing s and that crosses the potential plane of splitting through the reinforcement being developed, in^2
- s = maximum spacing of transverse reinforcement within ℓ_d center to center, in.
- K_{tr}
- n = number of bars or wires being developed along the plane of splitting

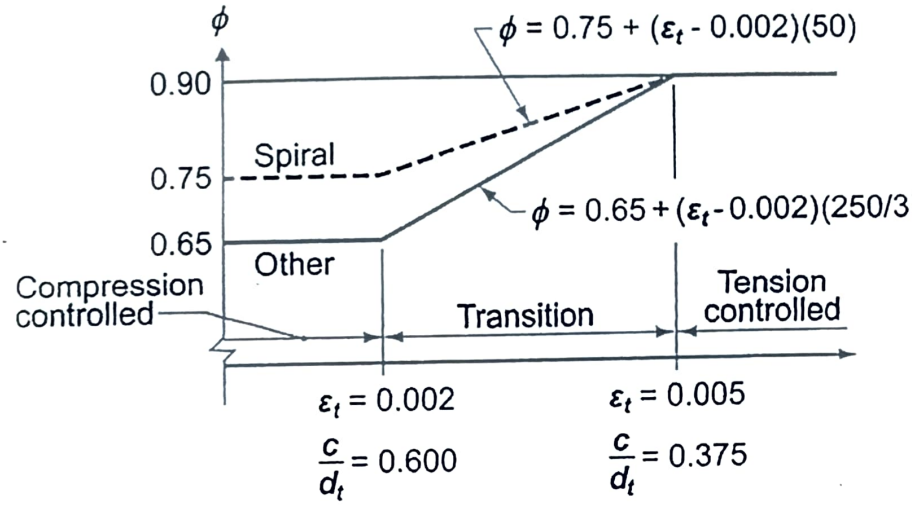
As a simplification, the designer is permitted to use $K_{tr} = 0$ even if transverse reinforcement is present.

Where $\dfrac{c_b + K_{tr}}{d_b} \le 2.5$

Development lengths for headed deformed bars in tension

Development lengths for headed deformed bars in tension

A. Development length ℓ_{dt} for headed bars	$\left(\dfrac{0.016 f_y \psi_e}{\sqrt{f'_c}} \right) d_b$
B. Modification factors applied to ℓ_{dt}	
ψ_e	
For epoxy-coated or zinc and epoxy dual-coated reinforcement	1.2
For other bars	1.0



Interpolation on c/d_t: Spiral $\phi = 0.75 + 0.15[(1/c/d_t) - (5/3)]$
Other $\phi = 0.65 + 0.25[(1/c/d_t) - (5/3)]$

Figure —*Variation of* ϕ *with net tensile strain in extreme tension steel,* ε_t, *and* c/d_t *for Grade 60 reinforcement and for prestressing steel.*

Development Length and Modification Factors for Hooked Bars

length is

$$\ell_{dh} = \left(\frac{f_y \psi_e \psi_c \psi_r}{50\lambda\sqrt{f'_c}} \right) d_b \qquad \text{(B)}$$

In Eq. (B) , the terms are defined values established as follows.

ψ_e = epoxy coating factor
For epoxy-coated or zinc and epoxy dual-coated reinforcement: 1.2
For uncoated and zinc-coated (galvanized) reinforcement: 1.0

ψ_c = cover factor
For No. 11 bar and smaller hooks with side cover (normal to plane of hook) $\geq 2\frac{1}{2}$ in. and for 90° hooks with cover on bar extension beyond hook ≥ 2 in.: 0.7
Other 1.0

ψ_r = confining reinforcement factor
For 90° hooks of No. 11 and smaller bars that are either (1) enclosed along ℓ_{dh} within ties or stirrups perpendicular to ℓ_{dh} at $s \leq 3d_b$ or (2) enclosed along the bar extension beyond hook including the bend within ties or stirrups perpendicular to the tail extension of the hook at $s \leq 3d_b$: 0.8
For 180-degree hooks of No. 11 and smaller bars enclosed along ℓ_{dh} within ties or stirrups perpendicular to ℓ_{dh} at $s \leq 3d_b$: 0.8
Other: 1.0

Note: The first tie or stirrup shall enclose the bent portion of the hook within $2d_b$ of the outside of the bend, and d_b is the nominal diameter of the hooked bar.

λ = lightweight concrete factor
For lightweight concrete: 0.75
Other: 1.0