PDHonline Course S250 (7 PDH)

Precast Concrete Connections

Instructor: Marvin Liebler, P.E.

2012

PDH Online | PDH Center

5272 Meadow Estates Drive
Fairfax, VA 22030-6658
Phone & Fax: 703-988-0088
www.PDHonline.org
www.PDHcenter.com

Precast Concrete Connections

1. GENERAL CONSIDERATIONS

1.1 DURABILITY [1,2] Note : Numbers in brackets refer to references in Section 4. below.

The three elements to be examined here are the durability of the concrete, durability of exposed metal parts, and durability of embedded metal parts. Durability of the concrete itself is provided in Chapter 4 of ACI 318-05 (hereafter referred to as the Code) which covers three major topics namely:

(1) Total air content for frost-resistant concrete
(2) Water/cement ratio by weight for special exposure conditions
(3) Maximum chloride ion content for corrosion protection of reinforcement

Exposed metal parts may be protected by these actions:

(1) Periodic inspections
(2) No chloride admixtures
(3) No water pockets
(4) Hot-dip galvanizing
(5) Stainless steel items

Embedded metal parts may be protected by:

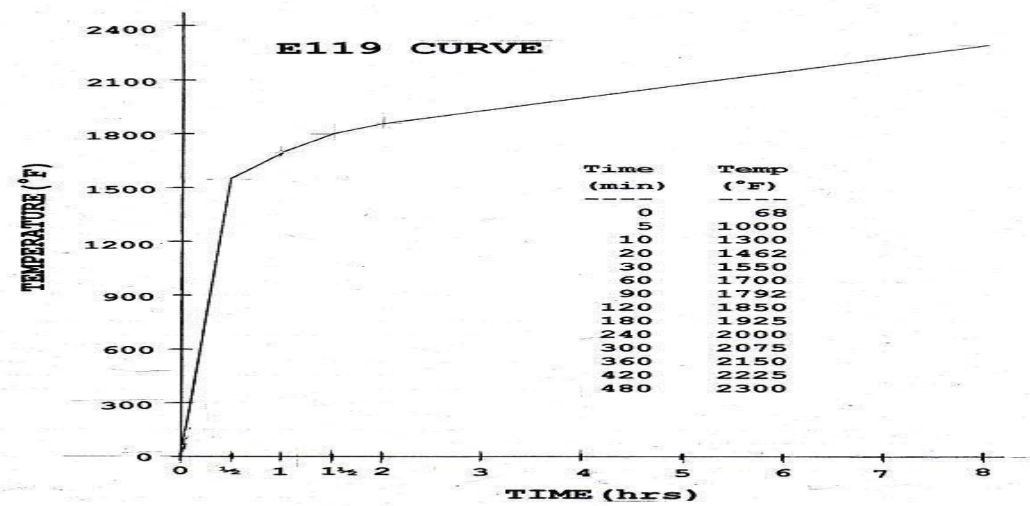
(1) Adequate cover
(2) No chloride admixtures
(3) Epoxy-coated bars
(4) Stainless steel items
(5) Galvanizing

1.2 FIRE RESISTANCE [3]

The connections which would result in structural failure during a fire should be protected the same as those for the structural frame. This is especially true for exposed, or near the surface, metal parts. The discussion below gives a general background on fire resistance of concrete and connections.

Fire resistance is the property of a component or frame to withstand fire or give protection from it.

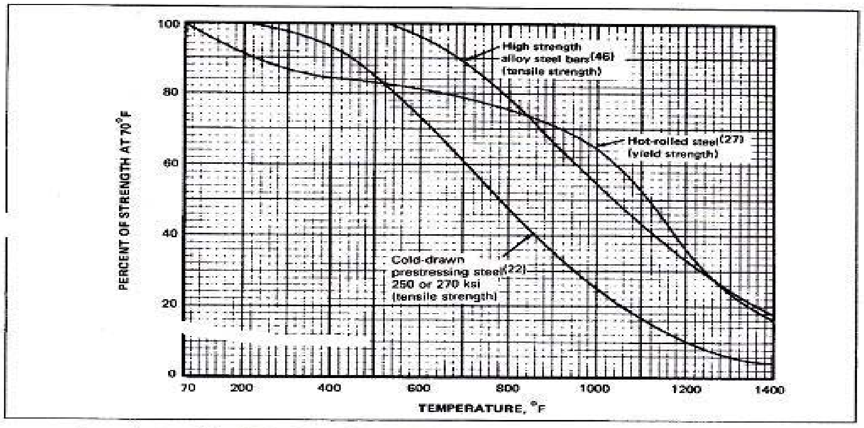
It is measured by the tests and conditions of ASTM E119 ("Standard Test Methods for Fire Tests of Building Construction and Materials"), with the standard temperature-time curve shown below. Notice that the starting temperature is 68°F.



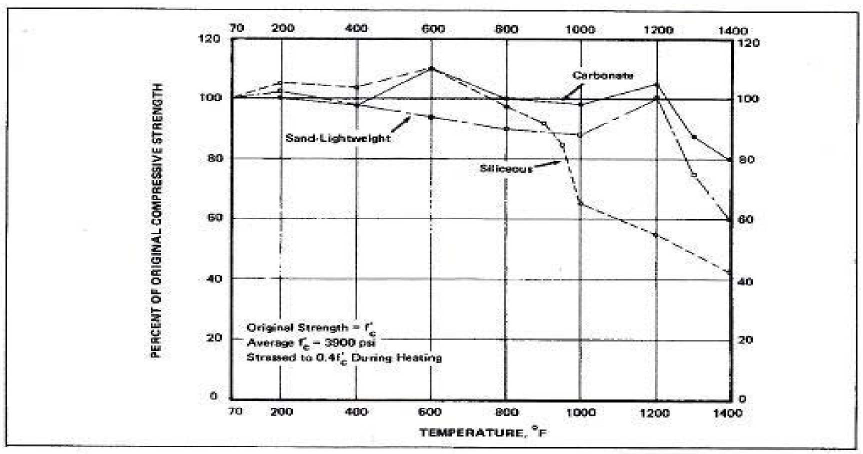
This fire represents combustion of about 10 lbf of wood (with a potential of 8000 BTU/lbf) per square foot of exposed area per hour of test, or 80000 BTU/hr/ft^2. By way of comparison, concrete has a thermal conductivity of 0.47-0.81 BTU/hr-ft-°F, while mild steel has a conductivity of 22.0-26.5 BTU/hr-ft-°F illustrating the much greater thermal resistivity of concrete.

Failure of the member under test (the end point) is due to the effects of the service load (maximum the structure is designed to support) plus the weakening of both the steel and concrete with temperature. Reference [3] gives the following data for modulus of elasticity and the two graphs below for strength of steel and concrete with temperature.

Es = 100%@68°F, 90%@600°F, 85%@800°F, and 72%@1000°F.
Ec = 100%@68°F, 70%@400°F, <50%@800°F, and 30%@1200°F, showing that deflections increase with temperature.



Temperature-strength relationships for hot-rolled and cold-drawn steels.

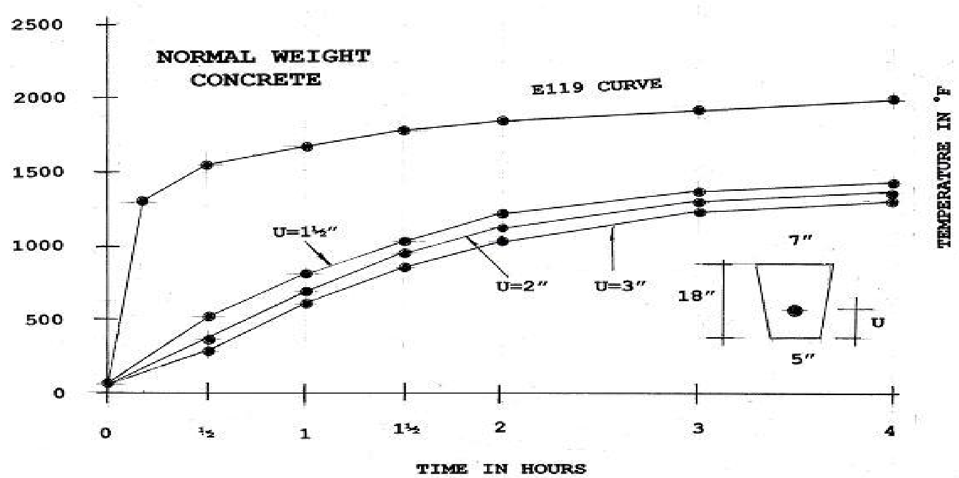


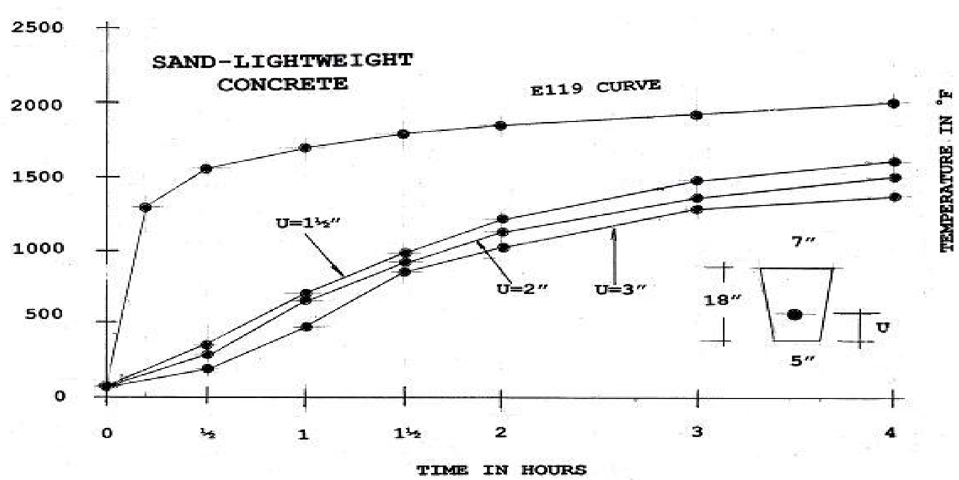
Compressive strength of concrete at high temperatures.

Carbonate concrete is made from aggregates consisting mainly of calcium or magnesium carbonate, for example, limestone or dolomite. Siliceous concrete is made from aggregates consisting mainly of silica or compounds other than calcium or magnesium carbonate.

Graphs on pages 76-78 of Reference [3] give the temperatures on vertical centerlines of stemmed units (joists) at ½,1,1½,2,3,and 4 hours into the E119 test, versus average stem width, with distance 'u' up from the bottom of the stemmed unit. Using this data, the

graphs below are constructed for temperature-time behavior of the centerline of the reinforcement for three (3) values of 'u', 1½, 2, and 3 inches. Note that a horizontal line at any particular temperature can be used to find the effectiveness of increasing 'u'. Note also that this is just one particular example and should not be extrapolated to other cases. It does, however, indicate the type of analysis that can be used if existing data is obtained.





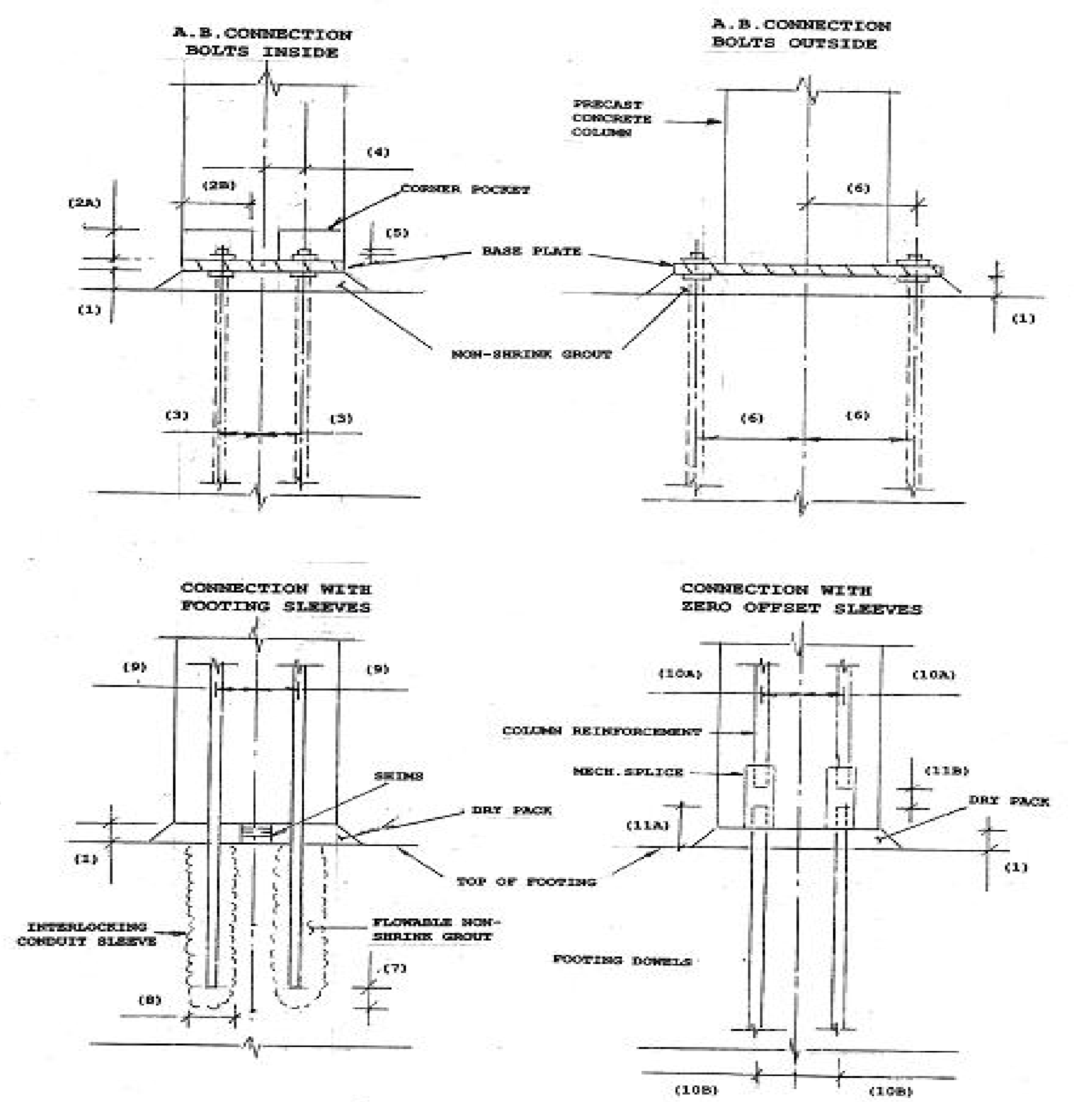
Connections with exposed metal, or metal close to the surface, should be protected to the same degree as the structural frame. Vermiculite cementitious spray coating, if tested for the application at hand, may be used.

1.3 TOLERANCES AND CLEARANCES

Tolerance is defined as the specified variation from a given drawing and/or specification value of a dimension, location, or alignment. There are tolerances on products, blockouts, openings, squareness, and erection.
Clearance is defined as the drawing and/or specification value of the distance between two elements to allow for product and erection tolerances, as well as to provide for sufficient space for carrying out connection operations, such as welding and bolt tightening. Clearances generally should be as large as possible.

Following below are four (4) types of column-foundation connections with suggestions as:

(1) This clearance based on the combination of column length and foundation elevation tolerances.
(2A, 2B) Pocket dimension to clear nut (point-point) Plus wrench head.
(3) Use templates to set anchor bolts.
(4) Control bolt hole dimensions to match (3) above.
(5) Check for full nut engagement in specifying anchor bolt length.
(6) Match drill holes for epoxy anchors.
(7) Provide enough clearance for proper flow of grout.
(8) Diameter should be at least twice that of rebar bundle.
(9) Use template to control projecting column rebar.
(10A, 10B) Dimensions are critical.
(11A, 11B) Consult manufacturer for proper dimensions and tolerances.


A.B.CONNECTION
BOLTS INSIDE
(2A)
(2B)
(4)
CORNER POCKET
(5)
BASE PLATE
(1)
NON-SHRINK GROUT
(3)
(3)
A.B.CONNECTION
BOLTS OUTSIDE
PRECAST
CONCRETE
COLUMN
(6)
(1)
(6)
(6)
CONNECTION WITH
FOOTING SLEEVES
(9)
(9)
SHIMS
DRY PACK
(1)
TOP OF FOOTING
INTERLOCKING
CONDUIT SLEEVE
FLOWABLE NON-
SHRINK GROUT
(7)
(8)
CONNECTION WITH
ZERO OFFSET SLEEVES
(10A)
(10A)
COLUMN REINFORCEMENT
MECH. SPLICE
(11B)
DRY PACK
(11A)
(1)
FOOTING DOWELS
(10B)
(10B)

2. DETAILS

2.1 REINFORCING BAR DEVELOPMENT [2, 5]

Definitions :

Ab = nominal reinforcing bar (rebar) cross-sectional area, in.^2.

Abrg = net headed bearing area, in.^2.

Atr = total cross-sectional area of all transverse reinforcement within spacing s that cross reinforcement being developed, in.^2.

cb = minimum of ½ the center-to-center rebar spacing or the least overall concrete cover dimension measured to the center of the bar.

cover = for precast other than walls:

(1) exposed to earth or weather :
 # 14 and #18 -> 2 in.
 #6 to #11 -> 1 ½ in.
 #5 + smaller -> 1 ¼ in.

(2) not exposed to weather or in contact with the ground, beams :
 primary reinforcement -> db but not less than 5/8 in. and need not exceed 1 ½ in.
 Ties and stirrups -> 3/8 in.

crack control = spacing of rebars closest to tension face not to exceed :
s=15(40000/fs)-2.5 cc but not greater than 12(40000/fs) where cc = least distance from surface of rebar to tension face. If there is only one bar, s is the width of the extreme tension face. fs may be taken equal to (2/3)fy.

db = nominal diameter of rebar, in.

fc′ = specified compressive strength of concrete, psi.

fy = specified rebar yield strength, psi.

fyt = specified yield strength of transverse reinforcement, psi.

Ktr = transverse reinforcement index = Atr(fyt)/1500(s)n. It can conservatively be set equal to zero.

ld = development length in tension of straight deformed rebar, in.

ldh = development length in tension for hooked (90° or 180°) hook. It is measured from the critical section to the start of the hook + inside radius of bend + one bar diameter,in. It is advisable to detail hooks precisely, avoid congestion, easily at corners.

ldt = development length in tension of headed deformed bar, in.

maximum beam rebar = strain of longitudinal steel >0.004 at nominal strength

n = number of bars.

s = center-center spacing of bars, longitudinal, transverse. Minimum clear spacing between bars in a layer = db and 1 in., unless specified by other factors.

structural integrity = In detailing of members and connections, members of a structure shall be effectively tied together.

λ = factor for concrete weight :
1.0 for normal weight concrete
1.3 for lightweight concrete.

Ψe = factor for rebar coating :
1.5 for epoxy coated anchors, closely spaced, or limited cover
1.2 for other epoxy-coated bars
1.0 for galvanized or no coating.

Ψs = factor for rebar size
0.8 for #6 and smaller
1.0 for #7 and larger.

Ψt = 1.3 for 12″ or more cast below bars
1.0 elsewhere.

Φ = 0.90 for tension-controlled sections
0.75 for compression controlled sections w/spiral reinforcement
0.65 for other compression-controlled
0.75 for shear
0.75 for torsion
0.65 for bearing on concrete

Proper internal or external anchorage of rebar in concrete is required for ductile behavior, i.e., bars yield before losing bond with concrete as opposed to brittle initial concrete failure. Internally this is done by the development concept, requiring a certain length and end configuration beyond the point of maximum bar stress. This development length (or anchorage) is necessary on both sides of the peak stress point. In general, do not terminate bars in a tension zone.
Externally, a mechanical device capable of developing the strength of the rebar without damage to concrete is allowed as anchorage.
Internal anchorage is done by any of three methods, namely straight bar extension, end hook (90° or 180°), or headed bar.

For a straight bar extension,

$$ld = \frac{3*fy*\psi t*\psi e*\psi s*\lambda}{40*\sqrt{fc'}*((cb+Ktr)/db)}*db \text{ where } Ktr = \frac{Atr*fyt}{1500*s*n}$$

and ld > = 12 in. and * denotes multiplication.
Note Ktr may be conservatively taken equal to zero.

Development length for a hook is :

$$ldh = \frac{0.02*\psi e*\lambda*fy}{\sqrt{fc'}} db \text{ where } \lambda = 1.3 \text{ for lightweight concrete}$$

and ldh > = 8*db and > = 6 in.

bar size	min. bend i.d.
#3 - #8	6*db
#9 - #11	8*db
#14, #18	10*db

where 90° bend has 12*db extension and 180° bend 4*db.

The requirements for headed bars are somewhat more complicated (Reference 5).

$$\text{By Code, } ldt = \frac{0.016*\psi e*fy}{\sqrt{fc'}} db >= 8*db \text{ and } 6in.$$

It is useful to construct tables showing the requirements of ACI 318-08.

ACI 318-08	condition
60000 psi	max fy
< #11	max bar size
normalweight	weight
4*Ab	min head area
2*db	min clear cover
4*db	min clear spacing
6000 psi	max fc'
8*db and 6 in.	min length

Example : Find development lengths for #8 bar for all three types .

Given :

- Abrg = 4*Ab
- Ab = 0.79 in.^2
- fy = 60000 psi
- fc' = 4000 psi
- bar center-center = 5in.
- clear cover = 2 in.
- bottom reinforcement
- normalweight concrete
- uncoated bars
- let (cb+Ktr)/db = 1.5, conservatively

Solution :

$$ld = \frac{3*fy}{40*\sqrt{4000}*1.5} db = 47.434*db, \text{ say } 48 \text{ in.}$$

$$ldh = \frac{.02*fy}{\sqrt{4000}} db = 18.974*db, \text{ say } 19 \text{ in.}$$

$$ldt = \frac{.016*fy}{\sqrt{4000}} db = 15.179*db, \text{ say 16 in.}$$

2.2 VOLUMETRIC CHANGES [2]

Three major factors contribute to volume change in reinforced concrete, namely creep, shrinkage, and temperature. These factors cause two major changes in member geometry, namely deflection and axial length.

Creep is defined as inelastic deformation under long-lasting loads with time, due to stresses in the elastic range, i.e., below approximately 0.5*fc'. It is due to water amount, water/cement ratio, curing conditions, relative humidity, size of member, age and duration of loading, and stress magnitude.

Shrinkage is defined as the decrease in time of the concrete volume. It is due to moisture loss in the concrete, hydration of the cement, and carbonation of the various products of hydration.

The Code treats the effect of creep and shrinkage together as:

$$\lambda = \frac{\xi}{1+50*\rho'}$$

λ = sustained load factor to multiply calculated deflection

Load Duration	ξ
3 months	1.0
6 months	1.2
1 year	1.4
➢ = 5 years	2.0

ρ'= compression steel ratio
  = As'/bd

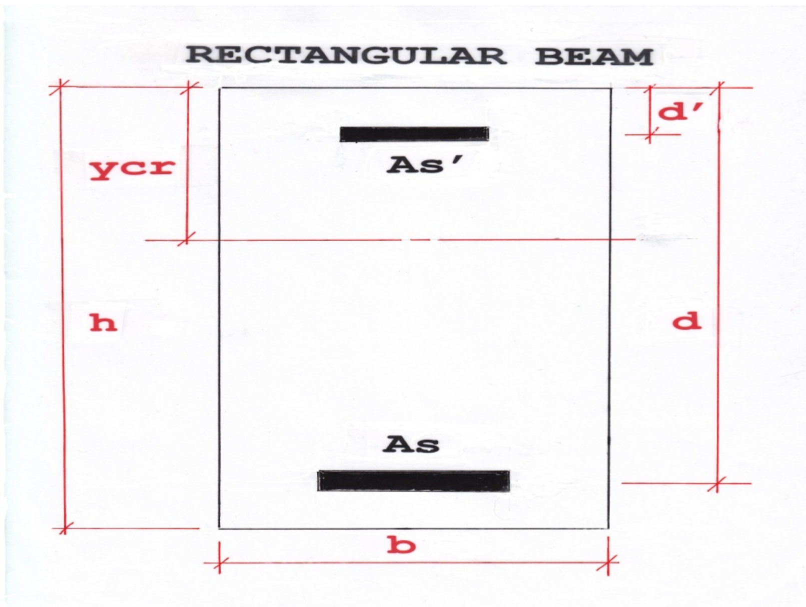
Example

Given : Rectangular beam with uniform service load of 1 kip/ft with the following parameters : b = 10", d = 18", d' = 2", h = 20", and pinned supports, L = 25 ft, fc' = 4000 psi,

tension reinforcement (As) = 3 #8 bars,
load duration = 5 years.

Find : Deflection at center span with compression reinforcement (As') = 0, 3 each of #4, #5, and #6 bars.

Solution :



Given a certain amount of compression reinforcement, the following procedure may be used to find the final(total) deflection.

(1) Find ycr, the cracked beam centroid, by equating first moments about the (unknown as of yet) ycr.
(b*ycr)*(ycr/2)+n*As'*(ycr-d') = n*As(d-ycr),
where n = Es/Ec.

ycr = (n/b)*(As+As')*(√radical-1) where

$$radical = 1 + \frac{2*b*(As*d+As'*d')}{n*(As+As')^2}$$

(2) Find Icr, the cracked moment of inertia.
Icr = b*ycr^3/3+n*As'*(ycr-d')^2+n*As*(d-ycr)^2

(3) Find Ie, effective moment of inertia, by the Code formula :

Ie=(Mcr/Ma)^3*Ig+(1-(Mcr/Ma)^3)*Icr where
Ma = maximum unfactored moment = (1/8)*w*L^2
where w = uniform service load
Mcr = moment at cracking = fr*Ig/yt
fr = modulus of rupture = 7.5*√fc′
Ig = gross moment of inertia = b*h^3/12
yt = ½ *h for a rectangular beam

(4) Find immediate deflection = 5*w*L^4/(384*Ec*Ie)

(5) Find λ = ξ/(1+50*ρ′) = sustained load factor

(6) Find final deflection = λ*immediate deflection

The results are:

As′(in.^2)	immediate deflection	final deflection
0.00	0.685″(L/438)	1.370″(L/219)
0.60(3#4)	0.667″(L/449)	1.144″(L/262)
0.93(3#5)	0.659″(L/455)	1.047″(L/286)
1.32(3#6)	0.650″(L/462)	0.951″(L/315)

Temperature change of volume is both bidirectional and reversible.
Temperature coefficients here are in./in/°F.
Let L = axial length of member after temperature change, L0 = initial length, T = final temperature, and T0 = initial temperature. The basic equation of length change is :
L/L0 = 1 + α*(T-T0)
where α = temperature coefficient of expansion = 0.55 x 10^(-5) to 0.75 x 10^(-5)deg(-1) for concrete and 0.65 x 10(-5) for steel.
Depending on the degree of restraint, this length change can result in large forces.
Example : For the 25′ beam above, find the length change for a 100 degree temperature change for an unrestrained beam, and the axial force induced by both ends pinned. Assume

both coefficients = 0.65*10^(-5)

Solution: ΔL = α*(T-T0)*L = 0.195″ (unrestrained)
The force generated is that which would stretch (or compress) the beam 0.195″.
Let steel area = 2*2.37 in.^2 =4.74 in.^2
concrete area = 10*20-4.74 = 195.26 in.^2
For an axial member, Δ = F*L/E*A ->
F = E*A*Δ/L
For the steel, F = 29000*4.74*0.195/300
F = 89.439 kip
and the concrete F = 3650*195.26*.195/300
F = 463.25 kip
Total force = 552.69 kip!
Reference [2] gives a chart showing design temperature changes versus geography. The 100 degree change used above corresponds to Northern Alaska and Canada. However,much of the U.S., including New York and parts of Texas show a fifty (50) degree temperature change, still yielding a formidable force. Efforts should be made to reduce restraint while maintaining stability and safety.

2.3 GROUT [6]

Reference [6] states that " The most important requirement for a grout that is intended to transfer loads to the foundation is that it has volume-change characteristics that result in complete and permanent filling of the space. Plain grouts consisting of cement, aggregate, and water do not have these characteristics." For this reason we cover only cementitious and epoxy nonshrink grouts.

An air-release system, which uses specially processed fine carbon, releases adsorbed air on contact with the mixing water and thus causes an increase in volume while grout is in the plastic state. This counteracts the normal settlement and drying shrinkage, and is insensitive to the alkali content of the cement used. It is used is many nonshrink cementitious grouts.

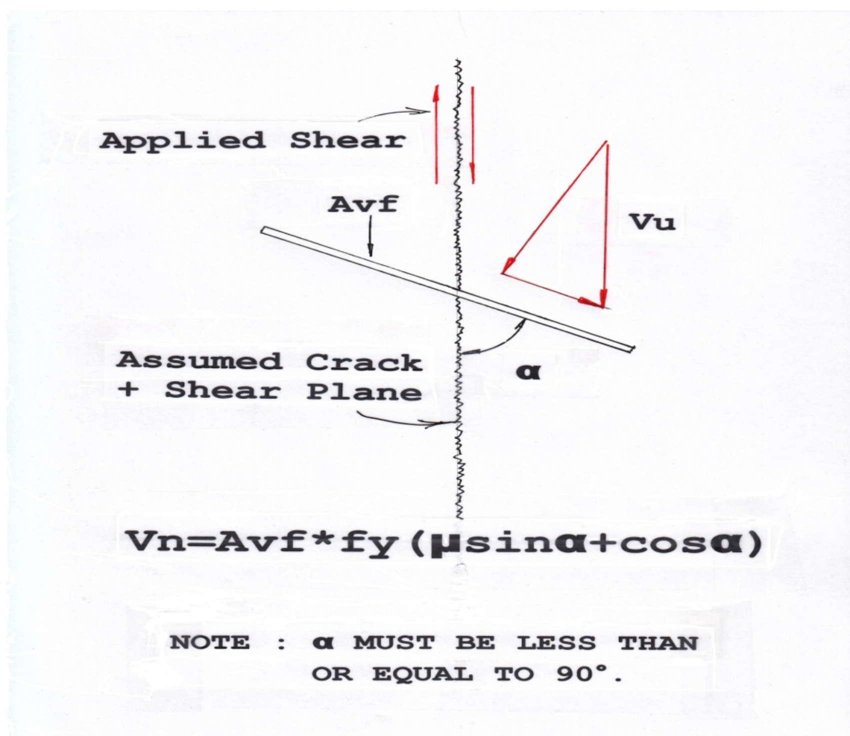
Epoxy grouts, consisting only of epoxy resin and hardener (converter, catalyst) , will shrink after placement. The grout usually contains specially blended aggregate, filler, or other ingredients to eliminate the shrinkage.

Detailing Considerations :

(1) Design so that the grout can be placed beneath the plate without trapping air or water in unvented edges or corners.

(2) If (1) is not possible, i.e., grout cannot be placed from one edge to the opposite side, provide air vent holes.

2.4 SHEAR-FRICTION [2]



The shear-friction concept is used when considering shear transfer occurs across:

(1) An existing or potential crack.

(2) An interface between different materials.

(3) An interface between concretes cast at different times.

It is assumed that a crack has occurred due to one of the above reasons.

Definition of Terms :

Ac	area of concrete resisting shear transfer, in.^2
Avf	area of shear-friction reinforcement, must be developed on each side of the shear plane, in.^2
bw	web width, in.
d	distance from extreme compression fiber to centroid of longitudinal tension reinforcement, in.
fc′	specified concrete strength of concrete, psi
fy	specified yield strength of reinforcement, psi
Vn	nominal shear strength, lbf (= Vu/**Φ**, Vu = factored load <= 0.2 fc′Ac and 800 Ac
α	angle of shear reinforcement
λ	= 1.0 for normalweight concrete = 0.85 for sand-lightweight concrete = 0.75 for all lightweight concrete
μ	coefficient of friction = 1.4**λ** for monolithic concrete = 1.0**λ** for fresh concrete cast against intentionally roughened hardened concrete = 0.6**λ** for fresh concrete cast against unroughened hardened concrete = 0.7**λ** for concrete anchored to as-rolled structural steel by headed studs or rebar

2.5 EPOXY ANCHORS [7]

These anchors are post=installed (after concrete has set) and require drilling in hardened concrete to the appropriate diameter and depth. They have the strong advantage of match drilling to holes in components in the field, and are available in diameters of 3/8″ to 1-1/4″. ASD (working) loads are available of over 30 kip/bolt in shear only, or tension only loads exceeding 75 kip/bolt, with proper material, diameter, depth, spacings, and edge distances.

The resin and hardener are put into the cleaned hole either by a mixing gun or glass cartridge which is broken upon inserting the bolt.

Several companies, such as Hilti, Simpson Strong-Tie, and Red Head manufacture adhesive anchoring systems. Here one of Hilti's products, "HIT-HY 150 MAX Adhesive Anchoring System", is studied., although the principles apply to the other brands. Each manufacturer does, however, have their own test results.

The bonding of anchor to concrete, using ASD (Allowable Stress Design) is covered in general in reference [7] on pages 134-136, with load tables on page 179, and factors on pages 188-190.
The strength of the system is dependent upon:

(1) anchor diameter
(2) anchor depth in concrete
(3) bending of bolt for load not applied at concrete surface
(4) bolt spacings
(5) anchor edge distance for tension
(6) anchor edge distance for shear perpendicular towards edge
(7) anchor edge distance for shear parallel to or away from edge
(8) combined tension and shear interaction
(9) bolt strength

Adjustment factors, after selection of bolt diameter, depth, and resolution of bending, (items (1) to (3)), determination of items (4) to (7), are given as:

f_{AN} = spacing factor for tension and/or shear

f_{RN} = adjustment factor for edge distance, tension

f_{RV1}= adjustment factor for shear perpendicular or towards edge

f_{RV2}= adjustment factor for shear parallel to or away from edge

The following equations apply to all conditions:

$s > s_{min}$

$c > c_{min}$

where:

s = spacing of anchors, in.

c = edge distance, in.

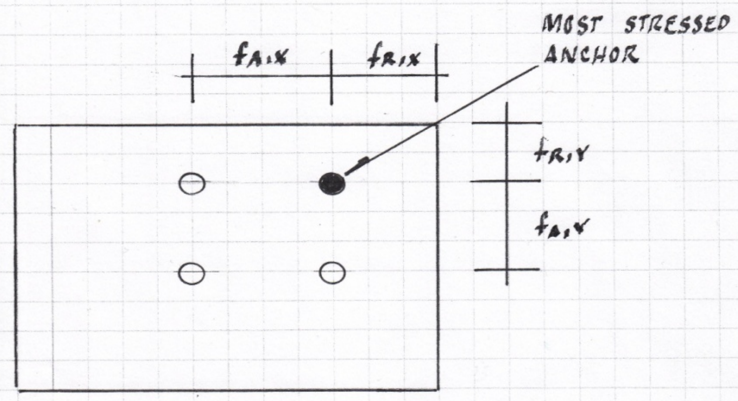
s_{min} = minimum spacing, in.

s_{cr} = spacing which does not require a reduction factor (additional spacing is discounted)

c_{min} = minimum edge distance

c_{cr} = edge distance which does not require a reduction factor (additional edge distance discounted)

h_{ef} = embedment depth cited in table for bond strength



f_A = SPACING FACTOR
f_R = EDGE FACTOR

$T_{ALLOWABLE} = f_{A,X} * f_{R,X} * f_{A,Y} * f_{R,Y} * T_{TABLE}$

T_{TABLE} = LESSER OF ALLOWABLE BOND + ALLOWABLE STEEL TABLES.

$V_{ALLOWABLE} = f_{A,X} * f_{R,X} * f_{A,Y} * f_{R,Y} * V_0$

V_0 = LOWEST OF (1) ALLOWABLE BOND TABLE
(2) ALLOWABLE STEEL TABLE
(3) V_{REC} FOR BOLT BENDING

Tables from reference [7] are shown below for normal weight of concrete. All values are based on holes drilled with carbide bit, cleaned with compressed air, wire brushed, and provided with sufficient resin and so as to be uniformly extruded upon placement of anchor.

Anchor Diameter in (mm)	Embedment Depth in (mm)	HIT-HY 150 MAX Allowable Bond/Concrete Capacity Tensile f'c = 2000 psi (13.8 MPa) lb (kN)	Tensile f'c = 4000 psi (27.6 MPa) lb (kN)	Shear f'c = 2000 psi (13.8 MPa) lb (kN)	Shear f'c = 4000 psi (27.6 MPa) lb (kN)
3/8 (9.5)	1 3/4 (44)	725 (3.2)	1155 (5.1)	1675 (7.5)	2360 (10.5)
	3 3/8 (86)	2110 (9.4)	3055 (13.6)	3155 (14.0)	4460 (19.8)
	4 1/2 (114)	2150 (9.6)	3055 (13.6)	4855 (21.6)	6860 (30.5)
1/2 (12.7)	2 1/4 (57)	1385 (6.2)	2090 (9.3)	2750 (12.2)	3890 (17.3)
	4 1/2 (114)	4000 (17.8)	4980 (22.2)	5610 (25.0)	7935 (35.3)
	6 (152)	4705 (20.9)	4980 (22.2)	8635 (38.4)	12210 (54.3)
5/8 (15.9)	2 7/8 (73)	1940 (8.6)	2730 (12.1)	4095 (18.2)	5790 (25.8)
	5 5/8 (143)	5955 (26.5)	8410 (37.4)	8760 (39.0)	12395 (55.1)
	7 1/2 (190)	7320 (32.6)	8410 (37.4)	13495 (60.0)	19080 (84.9)
3/4 (19.1)	3 3/8 (86)	2625 (11.7)	4295 (19.1)	6110 (27.2)	8635 (38.4)
	6 3/4 (172)	6460 (28.7)	9985 (44.4)	12615 (56.1)	17840 (79.4)
	9 (229)	11175 (49.7)	11175 (49.7)	19430 (86.4)	27470 (122.2)
7/8 (22.2)	4 (101)	3375 (15.0)	5300 (23.6)	7670 (34.1)	10840 (48.2)
	7 7/8 (200)	9910 (44.1)	14815 (65.9)	17175 (76.4)	24290 (108.0)
	10 1/2 (267)	14385 (64.0)	15345 (68.3)	26440 (117.6)	37390 (166.3)
1 (25.4)	4 1/2 (114)	5210 (23.2)	6570 (29.2)	9990 (44.4)	14120 (62.8)
	9 (229)	11595 (51.6)	17475 (77.7)	22435 (99.8)	31720 (141.1)
	12 (305)	17340 (77.1)	18685 (83.1)	34535 (153.6)	48830 (217.2)
1 1/4 (31.8)	5 5/8 (143)	6985 (31.1)	9935 (44.2)	13180 (58.6)	18640 (82.9)
	11 1/4 (286)	18345 (81.6)	30085 (133.8)	35050 (155.9)	49570 (220.5)
	15 (381)	25575 (113.8)	30085 (133.8)	53960 (240.0)	76300 (339.4)

Allowable Steel Strength for HAS Rods

Anchor Diameter in. (mm)	HAS-E Standard ISO 898 Class 5.8	
	Tensile lb (kN)	Shear lb (kN)
3/8 (9.5)	2640 (11.7)	1360 (6.0)
1/2 (12.7)	4700 (20.9)	2420 (10.8)
5/8 (15.9)	7340 (32.6)	3780 (16.8)
3/4 (19.1)	10570 (47.0)	5445 (24.2)
7/8 (22.2)	14385 (64.0)	7410 (33.0)
1 (25.4)	18790 (83.6)	9680 (43.1)
1-1/4 (31.8)	29360 (130.6)	15125 (67.3)

Adjustment Factors:

Spacing for tension and shear

$s_{min} = 0.5 \cdot h_{ef}$

$s_{cr} = 2.0 \cdot h_{ef}$

$f_{AN} = 0.165 \cdot (s/h_{ef}) + 0.67$

Tension edge distance

$c_{min} = 0.5 \cdot h_{ef}$

$c_{cr} = 1.5 \cdot h_{ef}$

$f_{RN} = 0.4 \cdot (c/h_{ef}) + 0.4$

Shear where hef < 9.0*nominal bolt diameter

cmin = 0.5*hef

ccr = 2.5*hef

Perpendicular to edge

f_{RV1} = 0.415*(c/hef)-0.0375

Parallel to or away from edge

f_{RV1} = 0.275*(c/hef)+0.312

Shear where hef >= 9.0*nominal bolt diameter

cmin = 0.5*hef

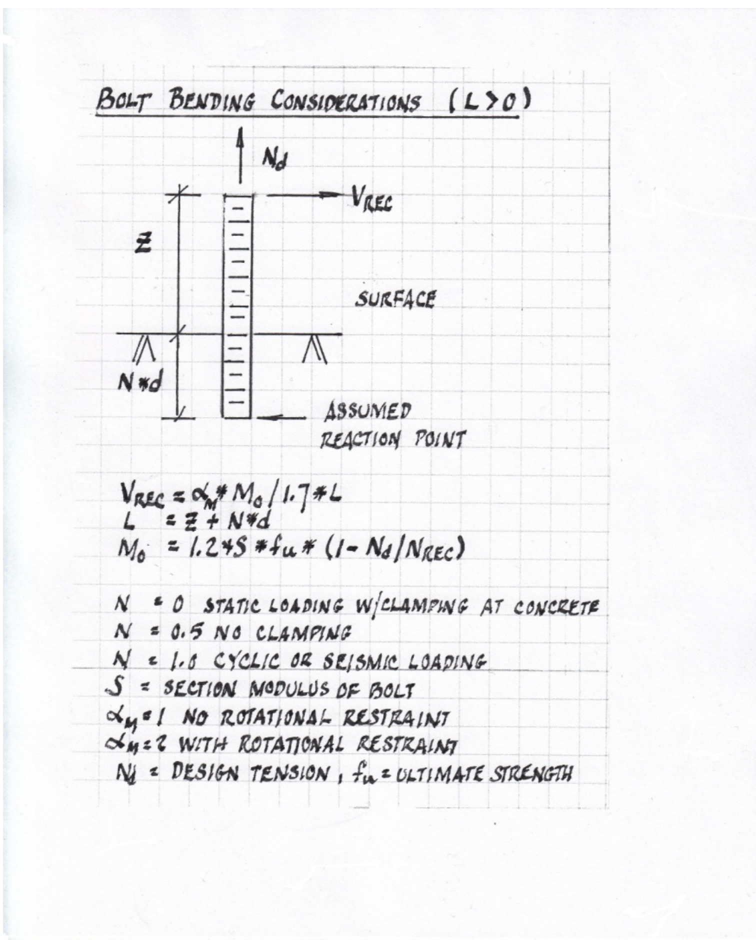
ccr = 2.0*hef

Perpendicular to edge

f_{RV2} = 0.554*(c/hef)-0.107

Parallel to or way from edge

f_{RV2} = 0.366*(c/hef)+0.267



3. SUPPORT STRUCTURES

3.1 Corbels [2,8,9,10]

Corbels are short cantilevers with shear span to depth ratios less than or equal to one. They are designed by special Code provisions (ACI 318, Section 11.9) as distinct from other longer cantilevered beams.

The corbel may fail by :

(1) Shearing along the interface between the supporting column or wall and the corbel.
(2) Yielding or pullout of the tension tie.
(3) Crushing or splitting of the compression strut.
(4) Localized bearing or shearing under the support plate.

The safety of the total structure depends on the sound design, detailing, and construction of the corbel, necessitating a detailed and thorough approach. The design method here is divided into nine parts, as:

3.1.1 Definition of Terms
3.1.2 Vu and Shear Span (av)
3.1.3 Nuc, d, and Mu
3.1.4 Shear-Friction Design (Avf)
3.1.5 Reinforcement to Resist Nuc (An)
3.1.6 Reinforcement to Resist Moment (Af)
3.1.7 Primary Tension Reinforcement (Asc)
3.1.8 Closed Stirrups or Ties (Ah)
3.1.9 Details

3.1.1 Definition of Terms

Af = area of reinforcement in corbel resisting factored moment, in.^2
Ag = gross area of supported beam, in.^2
Ah = total are of shear reinforcement parallel to primary tension reinforcement in a corbel, in.^2
An = area of reinforcement in a corbel resisting tensile force, Nuc, in.^2
As = area of longitudinal steel in supported beam, in.^2
Asc = area of primary tension reinforcement in a

corbel, in.^2

av	=	**shear span = distance from center of concentrated load to face of support for cantilevers, in.**
Avf	=	**area of shear reinforcement, in.^2**
bw	=	**web width, in. (also called 'b')**
d	=	**distance from the extreme compression fiber of concentrate to centroid of longitudinal reinforcement, in.**
fc'	=	**specified compressive strength of concrete, psi**
fy	=	**specified yield stress of reinforcement, psi**
h	=	**overall depth of member, in.**
L	=	**length of supported member, in.**
Mn	=	**nominal moment strength, lbf**
Mu	=	**factored moment of corbel section, lbf-in.**
Nuc	=	**factored horizontal tensile force applied at top of corbel, acting simultaneously with Vu, minimum of 0.2 Vu, max of 1.0 Vu, lbf**
Vn	=	**nominal shear strength, lbf**
Vu	=	**factored shear force at section, lbf**
α	=	**temperature coefficient of longitudinal concrete (and transformed longitudinal steel), in./in./°F**
β1	=	**.85, fc' <= 4000psi**
	=	**.85-.05(fc'-4000)/1000,4000 < fc'<=8000psi**
	=	**.65, fc' > 8000PSI**
Φ	=	**strength reduction factor, dimensionless .65 for bearing, all else .75 for corbels**

3.1.2 Vu and Shear Span (av)

There are three items that drive av, as:

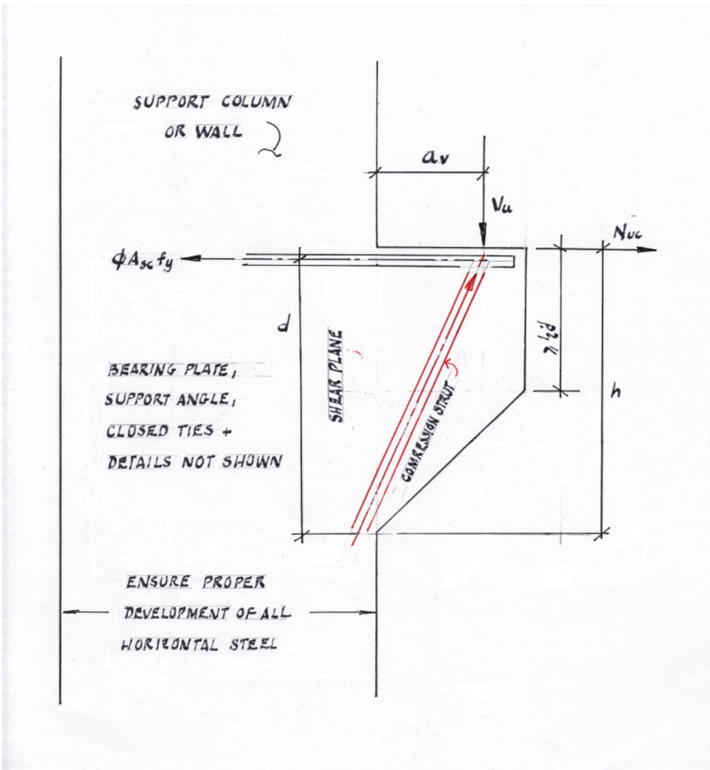
(1) Clearance of beam from support

(2) Allowable bearing on supported beam,
Vu <= Φ*.85*fc'*Abearing

(3) The outer edge of the bearing plate should not be less than two inches from the outer edge of the corbel.

3.1.3 Nuc, d, and Mu Calculations

The application should be carefully examined for all possible loads, including dead loads, live loads, wind, seismic and others, as well as lateral loads (perpendicular to the plane shown). Some of these may also involve sign reversal of Vu and/or Nuc. The procedure here assumes, however, that Vu and Nuc are the only loads and are in the directions shown below, with no lateral loads.



Vu = load factor (Code 9.2) times maximum working load, called factored load

Vn = nominal shear strength

= Vu divided by capacity reduction factor

Nuc is limited by the Code to be a minimum of 0.2 times Vu and a maximum of 1.0 times Vu.

This load is primarily due to creep, shrinkage, and temperature effects, and should be treated as a live load. The worst case is a newly cast beam fixed to two corbels at maximum temperature. See section 2.2 above, "Volumetric Changes", for temperature effects.

Reference 9 gives the following suggested values for ultimate shrinkage and creep, in the absence of specific data for local aggregates and conditions :
ultimate shrinkage strain=780x10^(-6)in./in.
ultimate creep = 2.35

The minimum depth of the corbel may be found from the Code restrictions on Vn, as:
Vn <= 0.2*fc'*bw*d and Vn < = 800*bw*d
This effectively limits the value of fc' which may be used to 4000 psi.
After solving for d, Mu may be determined as Mu = Vu*av+Nuc*(h-d), where moments are summed about the flexural reinforcement at the face of the column or wall support.

3.1.4 Shear-Friction Design (Avf)

As covered in section 2.4 above, this represents the design portion to correct for a possible vertical shear crack at the interface of the corbel and the supporting wall or column. This behavior is greater in the case of the corbel concrete and the support concrete cast at different times.

3.1.5 Reinforcement to Resist Nu (An)

This reinforcement may be calculated for the Equation for Nuc, as:
Nuc = Φ*An*fy

3.1.6 Reinforcement to Resist Moment (Af)

Here the traditional strength design for

beams is used. This is done in accordance with Code sections 10.2 and 10.3.

The basic equations for beam design are:

$M_n = f_y \cdot A_s \cdot (d - a/2)$ and $a = f_y \cdot A_s / (.85 \cdot f'_c \cdot b)$

These may be solved simultaneously to give:

$$A_{f,min} = \frac{.85 \cdot f'_c \cdot bd}{f_y}\left(1-\left(1-\frac{4 \cdot M_n}{1.7 \cdot f'_c \cdot b \cdot d^2}\right)^{.5}\right)$$

The maximum Af is given by the steel strain = .004 (>fy/Es) to give steel yield before concrete crushing.This may be calculated as:

$$A_{f,max} = \frac{2.55}{7} \cdot (f'_c/f_y) \cdot \beta_1 \cdot b_w \cdot d$$

Minimum reinforcement $= 3 \cdot (f'_c)^{(1/2)} b_w \cdot d/f_y$.

3.1.7 Primary Tension Reinforcement (Asc)

(1) $A_{sc} \geq A_f + A_n$

(2) $A_{sc} \geq (2/3) \cdot A_{vf} + A_n$

(3) $A_{sc}/bd \geq 0.04 \cdot (f'_c/f_y) \cdot b_w \cdot d$

3.1.8 Closed Stirrups or Ties

Ah must be greater than or equal to $0.5 \cdot (A_{sc} - A_n)$, and must be distributed with (2/3) adjacent to the primary tension reinforcement.

3.1.9 Details

(1) Choose practical bar sizes, having appropriate development, spacing, and cover.

(2) Design nose angle and welding of Asc to angle, using AWS D1.4.

(3) Design bearing plate welded to angle, ensuring that end deflection of beam maximum load and creep does not touch nose corner.

(4) Design tie framing bars.

EXAMPLE

Design an interior corbel with the following characteristics :

(1) Dead Load = 20 kip, Live Load = 50 kip.
(2) Support = 14 in. square column.
(3) All concrete = normalweight, 4000 psi.
(4) All steel = Grade 60.
(5) Corbel cast monolithically with support column.
(6) Allowance made otherwise for longitudinal contraction due to shrinkage and temperature.
(7) Allow two (2) inches nominal clearance at beam end from column to accommodate beam tolerance and column drift.
(8) Place outer edge of bearing plate two (2) inches from outer edge of corbel .
(9) Supported beam is 25 feet in length, 14″ wide, and 12″ deep.

3.1.2 Vu and Shear Span (av)

Vu = 1.4*DL + 1.6*LL = 108000 lbf
Abearing >= Vu/(.65*.85*4000) = 48.869 in.^2
Length of bearing plate =48.869/14 = 3.491 in.
Total length = 3.491 + 1.5 = 4.991 in., use 5 in.
av = 2 + ½*5 = 4.5 in.
This makes total corbel length = 2+5+2 = 9 in.

3.1.3 Nu, d, and Mu

Nuc = prescribed minimum of .2Vu = 21600 lbf
Vn = Vu/.75 = 144000 lbf
d <= Vn/(800*bw) = 12.587 in., use 14 in.
check av/d = 4.5/14 = =0.321 in. < 1 in., o.k.
Estimate Asc reinforcement = #5 bar and nose angle thickness = .75 in.
Thus h = 14+.625+.75 = 15.375 in.

Mu = Vu*av+Nuc(h-d) = 108000*4.5+21600*1.375 = 515700 lbf-in.
Mn = Mu/.75 = 687600 lbf-in.

3.1.4 Shear-Friction Design (Avf)

Avf = Vn/(fy*μ) = 1.714 in.^2

3.1.5 Reinforcement to Resist Nuc (An)

An = Nuc/(Φ*fy) = 0.480 in.^2

3.1.6 Reinforcement to Resist Mu (Af)

Using the formula in the text above,
Af = 0.851 in.^2
Check Af,max = 4.046 in.^2 > .851 in.^2,o.k.
Check Af,min = 0.620 in.^2 < .851 in.^2,o.k.

3.1.7 Primary Tension Reinforcement (Asc)

(1) Asc > = Af+An = 1.331 in.^2
(2) Asc >= (2/3)*Avf+An = 1.623 in.^2, governs
(3) Asc >= .04*(fc'/fy)*bw*d = 0.523 in.^2

3.1.8 Closed Stirrups or Ties (Ah)

Ah >= (Asc-An)/2 = 0.572 in.^2

3.1.9 Details

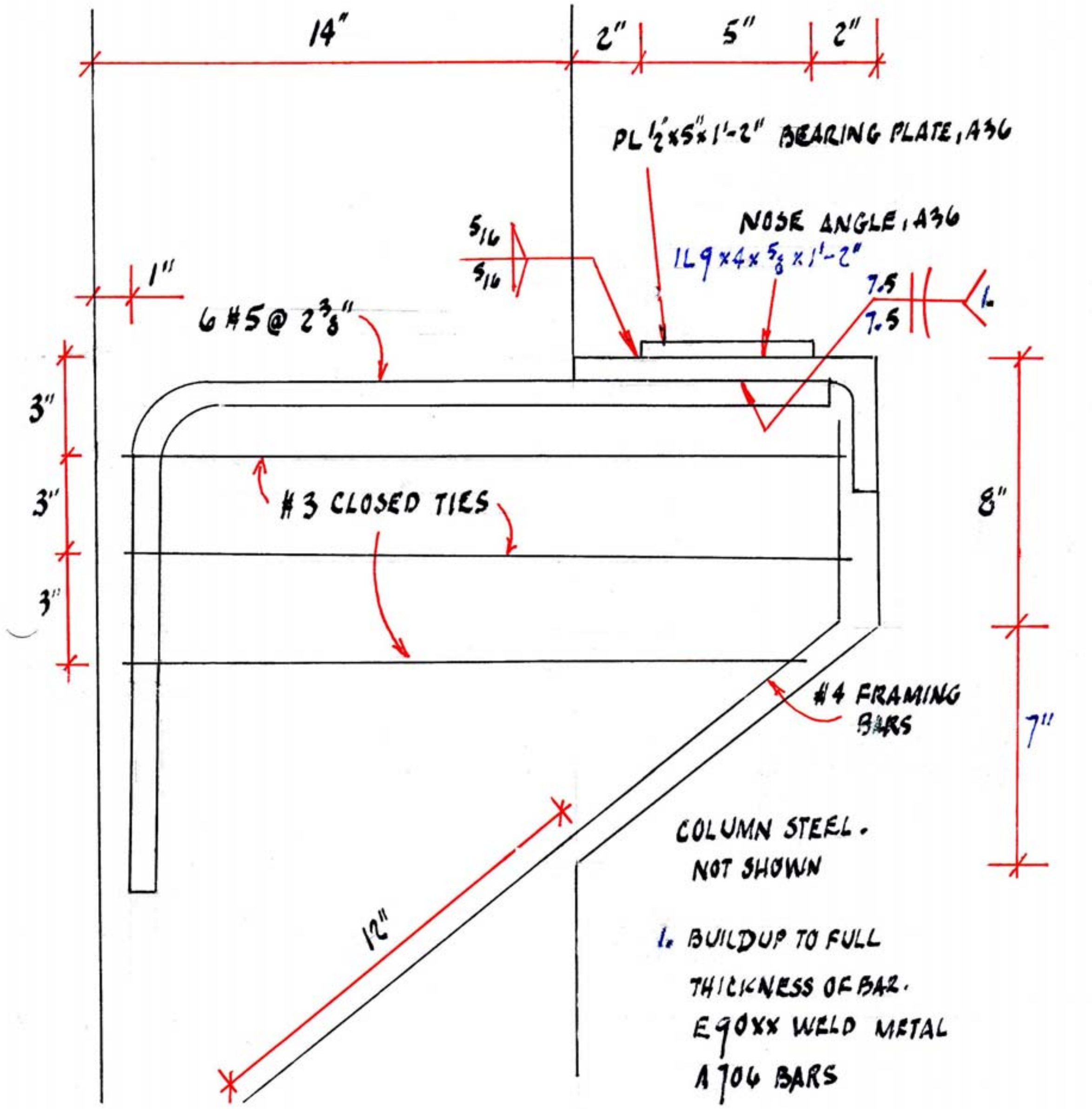
(1) Practical Bar Sizes
Development length for #5 bars with standard hooks = 11.858 in., which fits column width.
6 #5 bars=6x.31=1.86 in.^2>1.623 in.^2, o.k.
Spacing=(bw-2*cover-6*bar dia.-2*tie dia.)/5
= 1.600 in. > 1 in., o.k.

(2) Nose angle thickness>=5/8" for cover to Asc. Min length = 2"+5"+1/2" for bearing plate weldment, use 1L9x4x5/8x1'-2". Welding of the Asc bars to the nose angle is done by double flare-bevel-groove welds built up to the full thickness of the bar These welds have an allowable stress of 20 ksi for repetitive loads, using A706 Grade 60 bars, and using E90XX filler metal. The effective throat for #5 bars is .4*.5*.625 = .125 in. Thus the strength per inch for a single weld is .125*20 = 2.5ksi.

Required strength to develop a #5 bar is
fy*As = 60000*.31 = 18600 lbf
Conservatively, the minimum required length
= 18600/2500 = 7.440″
Use 7.5 in. length, centered under bearing plate.

(3) Bearing plate – minimum thickness to accommodate 5/16″ fillet weld = 3/8 in.
The deflection angle at the beam end under full load may be solved by the first moment-area theorem to get :
angle (radians) =(1/12*E*I)*w*L^3
w = distributed load = 2(20+50)/(12*25)
= 0.4667 kip/in.
L = 25*12 = 300 in.
E = 3605 ksi
I = (1/12)*14*12^3 = 2016 in.^4
Thus angle =0.1444 radians = 8.2755 degrees.
At full deflection, examining the triangle formed by the deflected beam and outside edge of the corbel, gives a downward deflection of the beam at the nose corner of 2*tan(8.2755)=.2909 in., use ½ inch plate.
Note that this thickness may have to be increased, depending upon long term deflection. Use PL ½″x5″x1′-2″, A36 steel.

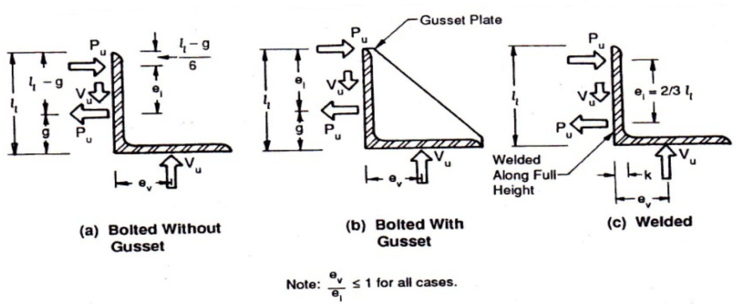
(4) Tie framing bars – good practice is to develop the bars into the support column.
From Section 2.1 above, development length = 12 in.



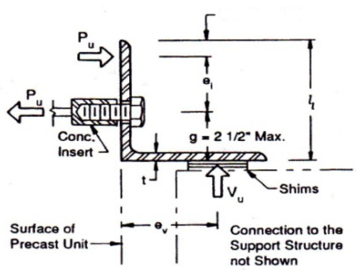
CORBEL EXAMPLE

3.2 CONNECTION ANGLES [1]

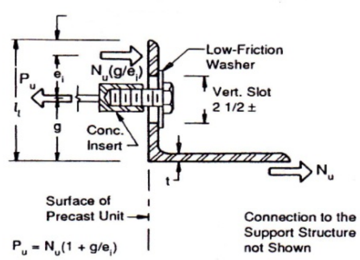
Angles supporting precast members generally are configured with one leg loaded in a perpendicular or parallel fashion, with the other leg resisting compression directly and tension through a cast-in place bolt, bolt and insert, or adhesive anchor. The figures below, from reference [1], represent these two cases.



Design Parameters for Connection Angles



Vertical Load on Angle



Horizontal Load on Angle

The text of reference [1] gives the following formulas for finding angle thickness :
For vertical (perpendicular) load :

thk = sqrt(4*Vu*ev/Φ*fy*b)

and bolt tension Pu = Vu*(ev/ei)

For parallel load:

thk = sqrt(4*Nu*g/Φ*fy*b)
and bolt tension Pu = Nu*(1+g/ei) where :

b	=	net length of angle, subtracting hole(s)′ diameter(s), in.
ei	=	distance from bolt hole center to far angle edge – (angle leg length-gage)/6, in.
ev	=	offset of vertical load from supporting angle face, in., use actual ev + angle thickness for design.
fy	=	specified yield strength of reinforcement, ksi
g	=	gage of angle, in.
Nu	=	factored horizontal load, kip
Vu	=	factored vertical load, kip
Φ	=	capacity reduction factor

The text has, for a vertical load Vu = 4 kips,
Design ev = 2.5″, actual ev = 2″, g = 2″, A36 steel,
5/8″ bolt hole, Φ = 0.9, 1L4″x4″x0′-4″.
The formulas above are solved to give:
thk = 0.605 in., use 5/8″
bolt tension = 6 kips
bolt shear = 4 kips

4.0 REFERENCES

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4.5 "Development and Splicing of Flexural Reinforcement Based on ACI 318-08", Jerry Spiker, Portland Cement Association, December 2008, www.cement.org/buildings/detailing_splash.asp

4.6 "Grouting between Foundations and Bases for Support of Equipment and Manufacturing", ACI 351.R-99, American Concrete Institute, www.concrete.org

4.7 "Hilti North American Product Guide", 2008 Edition, www.us.hilti.com

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4.9 "Prediction of Creep, Shrinkage, and Temperature Effects in Concrete Structures", ACI 209R-92 (reaffirmed 1997), www.concrete.org