Ankraj Tasarımında ACI 318-11 Yaklaşımı

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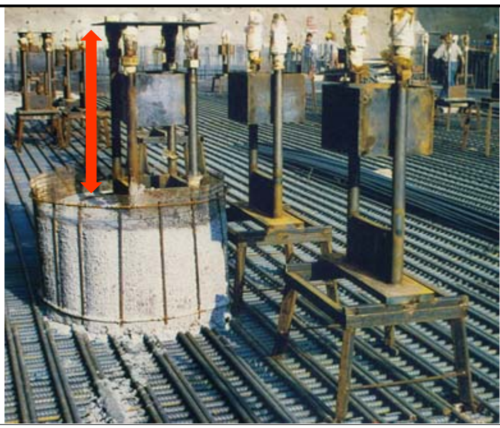
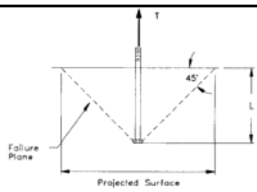
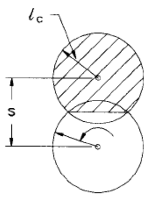
Bahar 2013 Dönemi Meslek İçi Seminerleri
21-22-23 Mayıs 2013



GÖRÜNTÜ

- 1970'li yılların ortalarında 45° Koni Methodu ilk kez kullanılmaya başlanmıştır.





- 1990'ların başlarında Beton Kapasitesi Yöntemi (Concrete Capacity Design-CCD) geliştirilmiştir.
- 1990'ların ortalarından itibaren ACI Komitesi **CCD** yöntemini benimsemiştir.

- ACI 318-14; 2014 yılı son çeyreğinde yayınlanacaktır.
- ACI 318-14 hakkında detaylı bilgileri ve değişiklikleri aşağıdaki adresten takip edebilirsiniz;

http://www.concrete.org/COMMITTEES/318public.htm

- **EK D—Betona Ankraj** başlığı anltında incelenmektedir.
- Sayfa ikiye bölünmüştür;

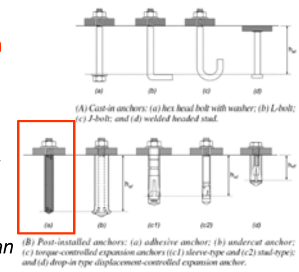
➡ Sol taraf: Yönetmelik (CODE)
➡ Sağ taraf: Açıklama (COMMENTARY)

ACI 318-11 Kapsamı

- Beton içinde önceden bırakılan ankrajlarla, sonradan betona bağlanan mekanik ve kimyasal ankrajlar bu yönetmelik kapsamındadır.
- *Yönetmelik kapsamı dışındaki ankrajlar:*

  Pnömatikle çalkılan çiviler

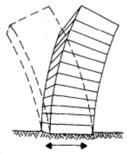
  Beton elemanın diğer tarafından çıkan ankrajlar
- Yorulma (fatigue) ve darbe (impact) yüklemesi kapsam dışındadır.



- Ankraj ve ankraj grupları elastik analizle elde edilen **arttırılmış (faktörlü)** yüklerin etkilerine göre tasarlanır.
- Deprem etkisinde plastik mafsal oluşacak bölgelere yapılan birleşimler yönetmelik kapsamı dışındadır.

  - Depremli hesap için;
    D.3.3. Sismik Tasarım Şartları bölümü incelenmelidir.

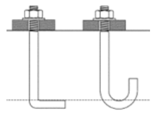
    Süneklikle ilgili şartlar sıralanmıştır.



Kanca ve 90° bükümlü ankrajlar

NCHRP (National Cooperative Highway Research Program)

- 90° bükümlü ve kanca ankrajların tam çekme kapasitelerini başlıklı ankrajlar kadar etkili kullanamadıkları tespit edilmiştir.
- Bu tip ankrajlarda çekme etkisinde bükülme bölgesinde akma görülmüştür.
- 90° bükümlü kancalarda çekme etkisinde bükülme bölgesinde betonda ezilmeler görülmektedir.
- Kancalar tam ankraj konisi oluşturamamaktadır.



- Yüksek dayanımlı bulonların bükülerek kanca haline getirilmesi malzeme özelliklerini negatif yönde etkilemektedir.

Yük Kombinasyonları

$$U = 1.4D$$

$$U = 1.2D + 1.6L + 0.5(L_r \text{ or } S \text{ or } R)$$

$$U = 1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (1.0L \text{ or } 0.5W)$$

$$U = 1.2D + 1.0W + 1.0L + 0.5(L_r \text{ or } S \text{ or } R)$$

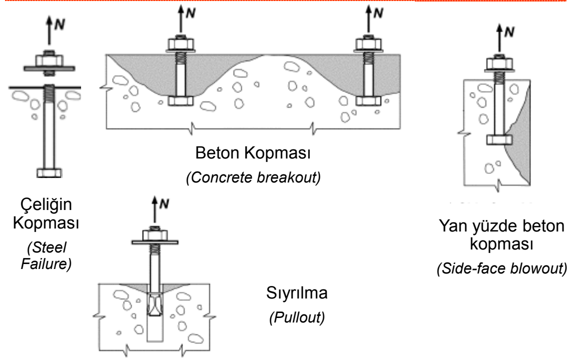
$$U = 1.2D + 1.0E + 1.0L + 0.2S$$

$$U = 0.9D + 1.0W$$

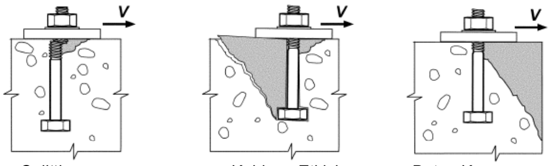
$$U = 0.9D + 1.0E$$

- D: Ölü Yük
- L:Hareketli yük
- Lr: Çatı hareketli yükü
- W: Rüzgar yükü
- E: Deprem kuvveti
- H: Zemin itkisi
- S: Kar yükü
- R: Yağmur yükü

Çekme Etkisinde



Kesme Etkisinde



Çeliğin Göçmesi *(Steel Failure)*

Kaldıraç Etkisi *(Concrete pryout)*

Beton Kopması *(Concrete breakout)*

Dayanım Azaltma Katsayıları (Ø)

Çelik Kapasitesi Hesabında

- Sünek Malzeme
- Çekme Etkisinde: 0.75
- Kesme Etkisinde: 0.65

- Gevrek Malzeme
- Çekme Etkisinde: 0.65
- Kesme Etkisinde: 0.60

DAYANIM

SÜNEKLİK

Dayanım Azaltma Katsayıları (Ø)

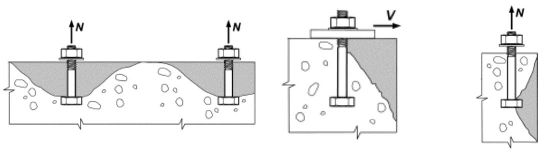
Beton Kapasitesi Hesabında

	A Durumu	B Durumu
1) Kesme Etkisinde	0.75	0.7
2) Çekme Etkisinde		
--> Başlıklı kayma çivisi ve başlıklı bulon	0.75	0.7
--> Mekanik ve Kimyasal Ankrajlar için		
Düşük Hassasiyetli Durum	0.75	0.65
Orta Hassasiyetli Durum	0.65	0.55
Yüksek Hassasiyetli Durum	0.55	0.45

A DURUMU
Ek donatı düzenlemesi yapılması durumudur.

B DURUMU
Ek donatı düzenlemesinin yapılmadığı durumdur.

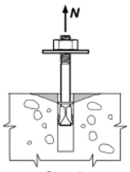
Ek donatı düzenlemesi hangi göçme modlarının dayanımı hesabında kullanılır?



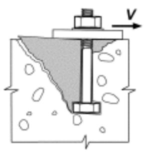
Beton Kopması *(Concrete breakout)*

Yan yüzde beton kopması *(Side-face blowout)*

Bu göçme modları dayanımları hesabında; ek donatı kullanılması durumunda A DURUMU, kullanılmaması durumunda B DURUMU katsayıları kullanılır.



Sıyrılma
(Pullout)



Kaldıraç Etkisi
(Pryout)

Bu göçme modları dayanımları hesabında;
Ek donatı düzenlemesi olsun veya olmasın B DURUMU katsayıları kullanılır.

Çekme Kuvveti İçin Tasarım Kriterleri

1) Çelik Ankraj Bulonu Tasarımı

Ø ile çarpmayı unutmayınız!!!

$$N_{sa} = nA_{se,N}f_{uta}$$

Bulon Sayısı

Etkili kesit Alanı [mm^2]

Kopma Dayanımı [Mpa]

$\boxed{f_{uta} < \min(1.9f_{ya};860\ \text{Mpa})}$

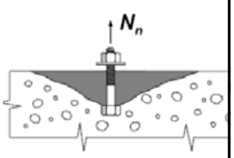
$$A_{se,N} = \frac{\pi}{4}\left(d_a - \frac{0.9743}{n_t}\right)^2$$

1mm'deki diş sayısı [1/p]

Diş aralığı tablosu

ÇAP(mm)	p(mm)
12	1.75
14	2
16	2
20	2.5
24	3
27	3
30	3.5

2) Çekmede Beton Kopması Dayanımı



Tek ankraj için

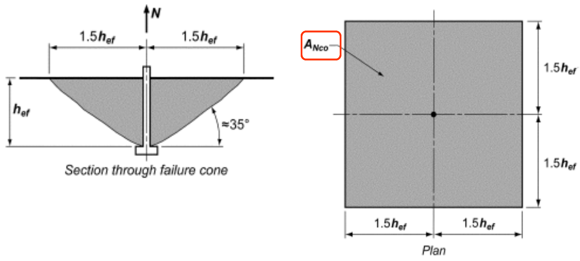
$$N_{cb} = \frac{A_{Nc}}{A_{Nco}} \Psi_{ed,N} \Psi_{c,N} \Psi_{cp,N} N_b$$

Ø ile çarpmayı unutmayınız!!!

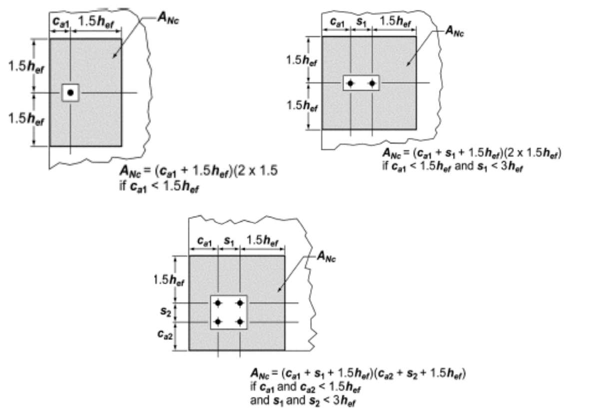
Ankraj Grubu için

$$N_{cbg} = \frac{A_{Nc}}{A_{Nco}} \Psi_{ec,N} \Psi_{ed,N} \Psi_{c,N} \Psi_{cp,N} N_b$$

Birbirine $3h_{ef}$'ten daha yakın olan ankrajlar grup olarak çalışır



$$A_{Nco} = (2 \times 1.5h_{ef}) \times (2 \times 1.5h_{ef}) = 9h_{ef}^2$$



$$N_{cb} = \frac{A_{Nc}}{A_{Nco}} \Psi_{ed,N} \Psi_{c,N} \Psi_{cp,N} N_b$$

Bir ankrajın çatlamış betondaki temel beton kopma dayanımı

$$N_b = k_c \lambda_a \sqrt{f_c'}\, h_{ef}^{1.5}$$

k_c : 10 (Beton içinde bırakılan ankrajlar için)
λ_a : Hafif beton için düzeltme katsayısı
f_c': Beton karakteristik dayanımı <= 70 Mpa (f_{ck})

Alternatif olarak;

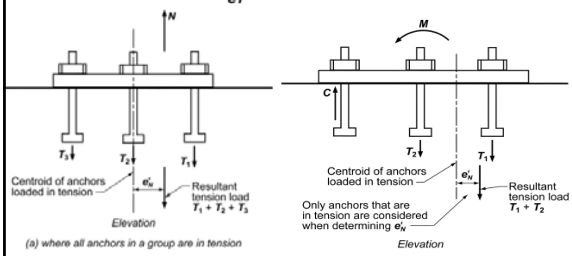
$280 \text{ mm} \le h_{ef} \le 635 \text{ mm}$ İçin;

$$N_b = 3.9\lambda_a \sqrt{f'_c}\; h_{ef}^{5/3}$$

Bu değerden büyük olamaz

Eksantrik çekme etkiyen ankraj grubu düzeltme katsayısı:

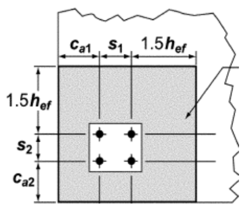
$$\psi_{ec,N} = \frac{1}{\left(1 + \dfrac{2e'_N}{3h_{ef}}\right)} \le 1$$



Kenar etkisi düzeltme katsayısı:

$c_{a,min} \ge 1.5h_{ef}$ ➡ $\psi_{ed,N} = 1.0$

$c_{a,min} < 1.5h_{ef}$ ➡ $\psi_{ed,N} = 0.7 + 0.3\dfrac{c_{a,min}}{1.5h_{ef}}$



Servis yükleri etkisinde betonun çatlamadığı gösterilebiliyorsa:

$$\psi_{c,N} = 1.25$$

$$N_{cb} = \frac{A_{Nc}}{A_{Nco}}\psi_{ed,N}\psi_{c,N}\psi_{cp,N}N_b$$

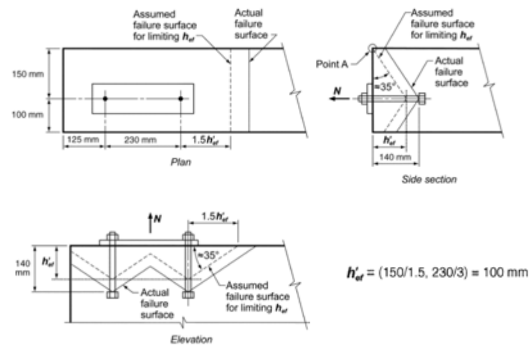
Bu katsayı betona sonradan yapılan ankrajlar için kullanılır.

ÖZEL DURUM-1:

- Bir ankrajın en az üç kenara **1.5h_{ef}**'ten yakın olması durumunda ;

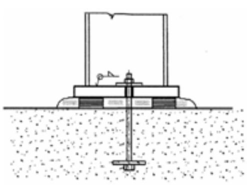
$$h'_{ef} = \max(c_{a,max}/1.5\ ;\ s_{max}/3)$$

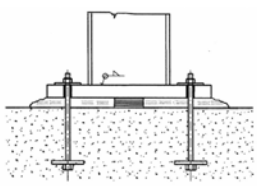
- Çekme etkisinde beton kopması hesabındaki tüm h_{ef}'ler yerine **h'_{ef}** kullanılır.



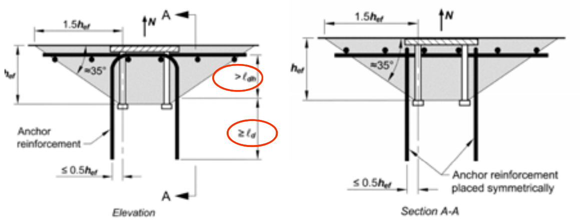
ÖZEL DURUM-2:

- Bulonun ucunda plaka kullanılması durumunda göçme yüzeyi, etkili plaka çevresinden itibaren 1.5h_{ef} dışarı ofsetlenerek belirlenir.
- Etkili plaka çevresi ankraj bulonu başlığına plaka kalınlığı eklenerek hesaplanır.



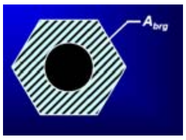


Çekmede Beton Kopması için Ek Donatı Düzenlemesi Yapılması



- Ankrajlardan **0.5h_{ef}** mesafe içinde kalan donatılar hesaba katılır.
- Ø=0.75 azaltma katsayısı kullanılır.
- $\O N_{cbg} = 0.75\ A_s\ f_{yd}$

3) Çekmede Betondan Sıyrılma Dayanımı



Ø ile çarpmayı unutmayınız!!!

$$N_{pn} = \psi_{c,P} N_p$$

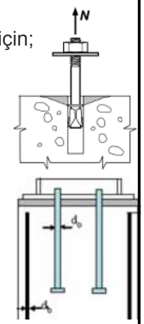
- Başlıklı kayma çivisi ve başlıklı ankraj bulonu için;

$$N_p = 8 A_{brg} f'_c$$

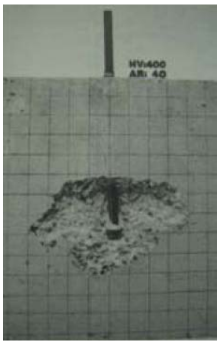
A_{brg} : Başlıklı saplama veya başlıklı bulon net ezilme alanı

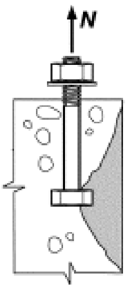
f'_c: Beton karaktereistik dayanımı (f_{ck})

- Servis yükleri altında çatlak oluşmadığı gösteriliyorsa; $\psi_{c,P} = 1.4$



4) Çekmede Yan Yüzde Beton Kopması Dayanımı



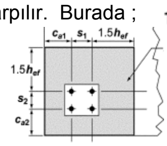


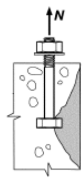
Ø ile çarpmayı unutmayınız!!!

- Derin ve kenara yakın ankrajlarda görülür. ($h_{ef} > 2.5c_{a1}$)

$$N_{sb} = (13 c_{a1} \sqrt{A_{brg}}) \lambda_a \sqrt{f'_c}$$

- Tek başlıklı ankraj için; $c_{a2} < 3c_{a1}$ ise N_{sb} değeri $(1 + c_{a2}/c_{a1})/4$ ile çarpılır. Burada ; $1.0 \le c_{a2}/c_{a1} \le 3.0$





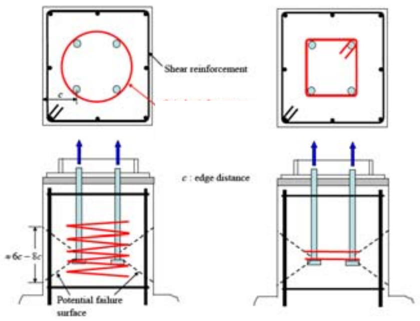
- Grup ankrajlarda;

$h_{ef} > 2.5c_{a1}$ ve ankrajlar arası mesafenin $6c_{a1}$'den küçük olması durumunda;

$$N_{sbg} = \left(1 + \frac{s}{6c_{a1}}\right) N_{sb}$$

s: Kenar boyunca uzanan ankraj sırasında dış uçlarda kalan bulonlar arası mesafe.

- Ankrajların kenara mesafelerinin en az 6d olması tavsiye edilir.



Bulon çevresine eklenecek olan spiral donatı veya etriye yan yüzeyden beton kopması kapasitesini ***arttırmaz!!!***

Kesme Kuvveti İçin Tasarım Kriterleri

1) Kesme Etkisinde Ankraj Bulonu Tasarımı

Ø ile çarpmayı unutmayınız!!!

Başlıklı kayma çivisi ankraj için;

$$V_{sa} = nA_{se,V}f_{uta}$$

Başlıklı ankraj bulonu için;

$$V_{sa} = n0.6A_{se,V}f_{uta}$$

$$A_{se,V} = \frac{\pi}{4}\left(d_a - \frac{0.9743}{n_t}\right)^2$$

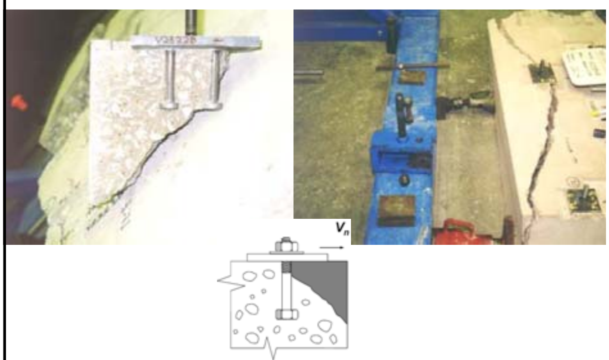
- Malzeme dayanımı ve etkili kesit alanıyla ilgili kurallar çekme etkisindeki bulon hesabıyla aynıdır.

ÖNEMLİ !!!!

- Ankraj plakası altında harç (grout) kullanılması durumunda ankraj bulonu kesme dayanımı **0.8** ile azaltılır.

NEDEN ?

2) Kesme Etkisinde Betonda Kopma Dayanımı



- Bir ankraj bulonuna kenara dik doğrultuda kesme etkimesi;

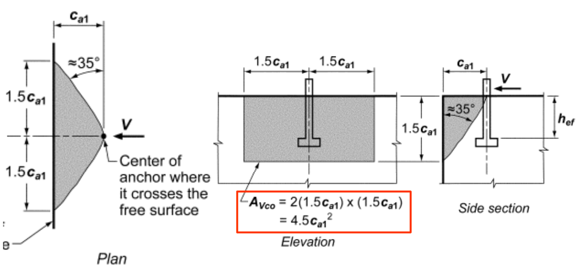
$$V_{cb} = \frac{A_{Vc}}{A_{Vco}} \Psi_{ed,V} \Psi_{c,V} \Psi_{h,V} V_b$$

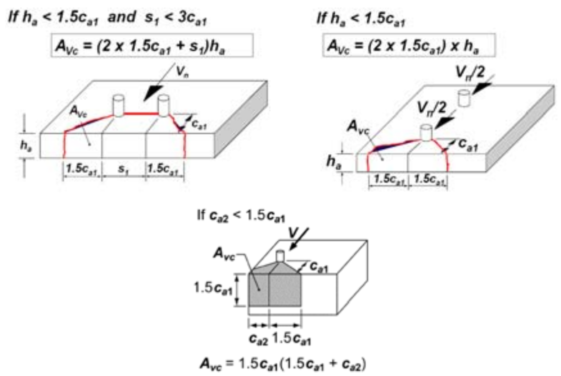
Ø ile çarpmayı unutmayınız!!!

- Ankraj bulonu grubuna kenara dik doğrultuda kesme etkimesi;

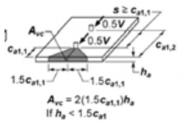
$$V_{cbg} = \frac{A_{Vc}}{A_{Vco}} \Psi_{ec,V} \Psi_{ed,V} \Psi_{c,V} \Psi_{h,V} V_b$$

Birbirine $3c_{a1}$'den daha yakın olan ankrajlar grup olarak çalışır

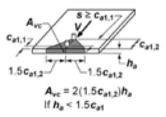




1.Durum:

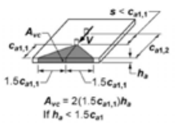


2.Durum:

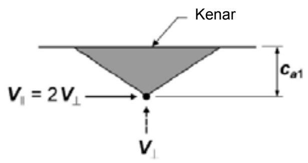


- $s \geq c_{a1,1}$ olması durumunda 1. ve 2. durumlar incelenir.

3.Durum:



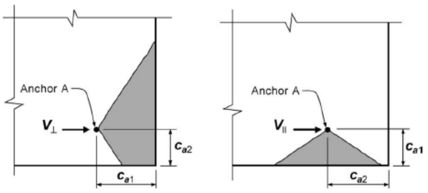
- $s < c_{a1,1}$ olması durumunda 3. durum incelenir.



- Beton kopması dayanımı, kenara paralel kesme kuvveti uygulanması durumunda;

$$\psi_{ed,V} = 1.0$$

alınarak kesme kuvveti kenara dik etkiyormuş gibi hesaplanan V_{cb} veya V_{cbg}'nin **2 katı** olarak alınır.



Köşede yer alan bulonlara kesme etkimesi durumunda;

- Kenara dik etkiyen kesme kuvveti
- Kenara paralel etkiyen kesme kuvveti

ayrı ayrı düşünülerek beton kopma kapasitesi hesaplanır.

$$V_{cb} = \frac{A_{Vc}}{A_{Vco}} \Psi_{ed,V} \Psi_{c,V} \Psi_{h,V} V_b$$

Tek ankrajın çatlamış betondaki beton kopma kapasitesi iki formülden elde edilen değerin **minimumudur.**

$$V_b = \left(0.6\left(\frac{\ell_e}{d_a}\right)^{0.2}\sqrt{d_a}\right)\lambda_a\sqrt{f'_c}\,(c_{a1})^{1.5}$$

$$V_b = 3.7\lambda_a\sqrt{f'_c}\,(c_{a1})^{1.5}$$

d_a : Bulon çapı

l_e: Akrajın yük taşıyan boyu (Beton içinde önceden bırakılan ankrajlar için h_{ef}'e eşittir.)

Üst sınır; $\ell_e \le 8d_a$

Başlıklı kayma çivisi veya başlıklı bulonların ;

max (10mm ; d_a/2) kalınlığındaki plakaya kaynaklanmaları durumunda;

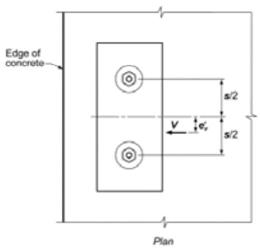
$$V_b = \left(0.66\left(\frac{\ell_e}{d_a}\right)^{0.2}\sqrt{d_a}\right)\lambda_a\sqrt{f'_c}\,(c_{a1})^{1.5}$$

Buna göre;

- En uzak sıra ankraja göre kapasite belirlenir.
- Bulonlar arası mesafe 65mm'den az olamaz.
- c_{a2}<1.5h_{ef} ise ek donatı düzeni gerekir.

Eksantrik kesme etkiyen ankraj grubu düzeltme katsayısı:

$$\Psi_{ec,V} = \frac{1}{1+\frac{2e'_V}{3c_{a1}}} \le 1$$

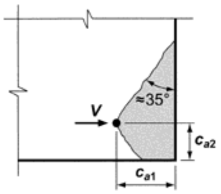


- Bir bulona eksantrisiteden dolayı daha fazla yük gelmesi olasılığından dolayı bu düzeltme katsayısı kullanılır.

Kenar etkisi düzeltme katsayısı:

$c_{a2} \geq 1.5c_{a1} \rightarrow \psi_{ed,V} = 1.0$

$c_{a2} < 1.5c_{a1} \rightarrow \psi_{ed,V} = 0.7 + 0.3\dfrac{c_{a2}}{1.5c_{a1}}$



Servis yükleri etkisinde betonun çatlamadığı gösterilebiliyorsa:

$\psi_{c,V} = 1.4$

$\psi_{c,V} = 1.0$ Donatısız çatlamış betondaki ankrajlar

$\psi_{c,V} = 1.2$ Donatılı çatlamış betondaki ankrajlar

$\psi_{c,V} = 1.4$ Donatılı ve max. 100mm aralıklı etriyeli çatlamış betondaki ankrajlar

$h_a < 1.5c_{a1}$ olan beton elemanlarda düzeltme katsayısı;

$$\psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} >= 1$$

h_a : Ankrajın bağlandığı beton elemanın kalınlığı. (Ankraj ekseni doğrultusunda)

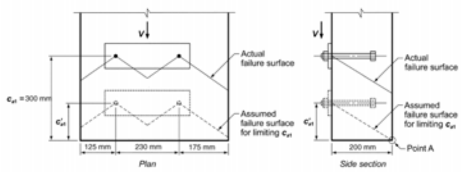
ÖZEL DURUM-1:

- Bir ankrajın en az üç kenara yakın olması durumunda ;

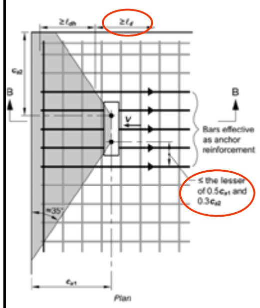
  c_{a2} ve $h_a < 1.5\ c_{a1}$ ise;

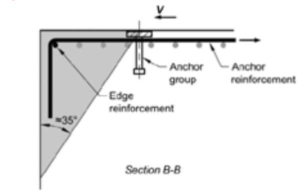
  $c'_{a1} = \max(c_{a,2\ max}/1.5\ ;\ h_a/1.5\ ;\ s_{max}/3)$

- Kesme etkisinde beton kopması hesabındaki tüm c_{a1}'ler yerine **c'_{a1}** kullanılır.



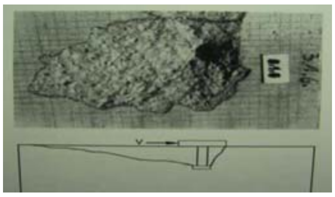
Kesmede Beton Kopması için Ek Donatı Düzenlemesi Yapılması





- Ankrajlardan **min(0.5 c_{a1}; 0.3c_{a2})** mesafe içinde kalan donatılar hesaba katılır.
- Ø=0.75 azaltma katsayısı kullanılır.
- $ØV_{cbg}$= 0.75 A_s f_{yd}

3) Kesme Etkisinde Betonda Kaldıraç Etkisi Dayanımı



Ø ile çarpmayı unutmayınız!!!

Tek ankraj için;

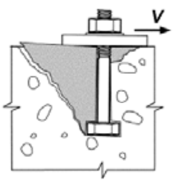
$$V_{cp} = k_{cp}N_{cb}$$

Ankraj grubu için;

$$V_{cpg} = k_{cp}N_{cbg}$$

k_{cp} = 1.0 → h_{ef} < 65 mm

k_{cp} = 2.0 → h_{ef} ≥ 65 mm



- **Kısa ve rijit** ankrajlarda etkilidir.

Çekme ve kesme göçme modu dayanımlarından **minimumları** ankraj bulonunun çekme ve kesme kapasitesini verir.

Çekme ve Kesme Kuvvetleri Etkileşimi

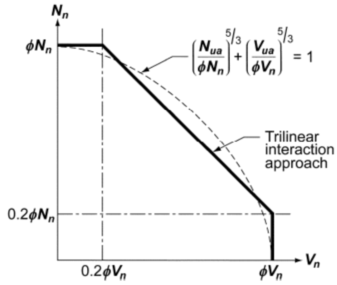
$V_{ua} \leq 0.2\phi V_n$ — Tam kapasite çekmeye çalışabilir

$N_{ua} \leq 0.2\phi N_n$ — Tam kapasite kesmeye çalışabilir

V_{ua} : Ankraja etkiyen faktörlü kesme kuvveti
N_{ua} : Ankraja etkiyen faktörlü çekme kuvveti

$V_{ua} > 0.2\phi V_n$ ve $N_{ua} > 0.2\phi N_n$

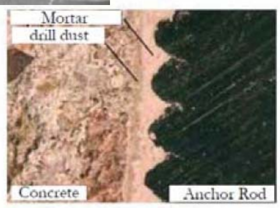
$$\frac{N_{ua}}{\phi N_n} + \frac{V_{ua}}{\phi V_n} \leq 1.2$$



Kimyasal Ankrajlar

- 21 günlük betona kimyasal ankraj uygulaması yapılabilir.
- Delik çapı: $1.5d_a$'dan fazla olmamalıdır.
- Minimum beton dayanımı 17MPa olmalıdır.
- Uygulama anında beton sıcaklığı minimum 10 derece olmalıdır.

- Betona delik açıldıktan sonra fırça ve pompa yardımıyla delik içindeki toz mutlaka temizlenmelidir.



$4d_a \leq h_{ef} \leq 20d_a$ için

- Tek ankraj için

$$N_a = \frac{A_{Na}}{A_{Nao}} \Psi_{ed,Na} \Psi_{cp,Na} N_{ba}$$

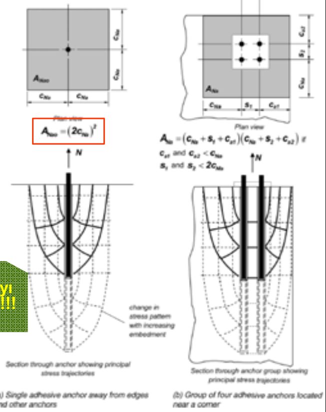
- Ankraj grubu için

$$N_{ag} = \frac{A_{Na}}{A_{Nao}} \Psi_{ec,Na} \Psi_{ed,Na} \Psi_{cp,Na} N_{ba}$$

$$c_{Na} = 10 d_a \sqrt{\frac{\tau_{uncr}}{7.6}}$$

Ø ile çarpmayı unutmayınız!!!

τ_{uncr} :Çatlamamış beton için yapışma dayanımı



$$N_{ba} = \lambda_a \tau_{cr} \pi d_a h_{ef}$$

TABLE D.5.5.2 — MINIMUM CHARACTERISTIC BOND STRESSES*†

Installation and service environment	Moisture content of concrete at time of anchor installation	Peak in-service temperature of concrete, °C	τ_{cr}, MPa	τ_{uncr}, MPa
Outdoor	Dry to fully saturated	79	1.4	4.5
Indoor	Dry	43	2.1	7.0

*Where anchor design includes sustained tension loading, multiply values of τ_{cr} and τ_{uncr} by 0.4.

†Where anchor design includes earthquake loads for structures assigned to Seismic Design Category C, D, E, or F, multiply values of τ_{cr} by 0.8 and τ_{uncr} by 0.4.

- Deprem yükleri etkisinde hesap yapılıyorsa;

τ_{cr} 0.8 ile
τ_{uncr} 0.4 ile çarpılacaktır.

- Eksantrik çekme etkisi düzeltme katsayısı:

$$\Psi_{ec,Na} = \frac{1}{\left(1 + \frac{e'_N}{c_{Na}}\right)} <= 1$$

- Kenar etkisi düzeltme katsayısı:

$c_{a,min} \geq c_{Na}$ ➡ $\Psi_{ed,Na} = 1.0$

$c_{a,min} < c_{Na}$ ➡ $\Psi_{ed,Na} = 0.7 + 0.3 \frac{c_{a,min}}{c_{Na}}$

- Çatlamamış beton için tasarlanan ankraj için düzeltme katsayısı:

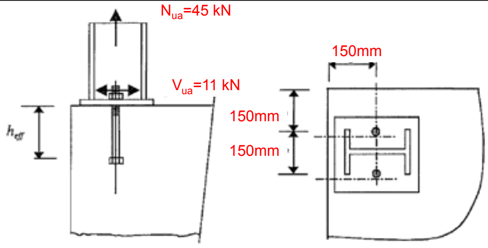
$c_{a,min} \geq c_{ac}$ ➡ $\Psi_{cp,Na} = 1.0$

$C_{ac} = 2h_{ef}$

$c_{a,min} < c_{ac}$ ➡ $\Psi_{cp,Na} = \frac{c_{a,min}}{c_{ac}}$

Birbirine $2c_{Na}$'dan daha yakın olan ankrajlar grup olarak çalışır

ÖRNEK ÇÖZÜMÜ



- fc'=25 Mpa
- Ankraj Bulonları M16 (5.8 Kalitesinde)
- hef=200mm

Ankrajın verilen **arttırılmış** yükler etkisindeki yeterli olup olmadığını belirleyelim...

ÇEKME DAYANIMININ BELİRLENMESİ

- Çelik Dayanımı

$N_{sa} = nA_{se,N}f_{uta}$ Ø=0.75 , ***Sünek çelik davranışı***

f_{uta}= 500 Mpa

$$A_{se,N} = \frac{\pi}{4}\left(d_a - \frac{0.9743}{n_t}\right)^2$$

p = 2mm (Diş aralığı)

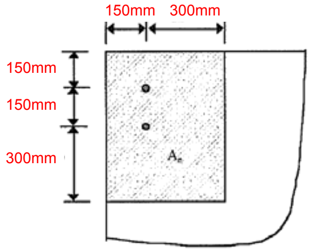
$A_{se,N}$= 155 mm^2

ØN_{sa}=0.75 *2*155*500= 116250 N = $\boxed{116.25 \text{ kN}}$

- Beton Kopma Dayanımı

$$N_{cbg} = \frac{A_{Nc}}{A_{Nco}} \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b$$

Ø=0.70 , Ek donatı düzenlemesi yoktur.



A_{Nco}= 9h_{ef}^2=9*200^2=360000mm^2

A_{Nc}= 600*450=270000mm^2

★ $\Psi_{ec,N}$=1 (Eksantrisite yoktur)

★ $\Psi_{ed,N}$ 'nin belirlenmesi:

$C_{a,min}$ =150mm < $1.5h_{ef}$ =300mm

$\psi_{ed,N} = 0.7 + 0.3\frac{c_{a,min}}{1.5h_{ef}}$ =0.85

★ $\Psi_{c,N}$ =1 (beton kenar ve köşeleri çatlak oluşması muhtemel bölgelerdir)

★ $N_b = k_c \lambda \sqrt{f'_c}\, h_{ef}^{1.5}$

k_c = 10

N_b=10*SQRT(25)*$200^{1.5}$=9*200^2=141421 N= 141.4 kN

★★★ $ØN_{cbg}$=0.70 * (270000/360000)* 1 * 0.85 * 1 * 141.4= $\boxed{\text{63 kN}}$

- Ankraj Sıyrılma Dayanımı

$$N_{pn} = \psi_{c,P} N_p$$

★ $\Psi_{c,P}$ =1 (beton kenar ve köşeleri çatlak oluşması muhtemel bölgelerdir)

★ $N_p = 8A_{brg} f'_c$

A_{brg}=292mm^2 (Strength Design of Anchorage to Concrete Tablo-2)

★★★ $ØN_p$= 0.70 * 2 * 1 * 8 * 292* 25= 81760 N= $\boxed{\text{81.76 kN}}$

- Yan Yüzde Beton Kopması Dayanımı

h_{ef}= 200 < $2.5c_{a1}$ = 2.5 * 150 = 375 şartı sağlandığından

★ Bu göçme modunun kontrolüne gerek yoktur.

➡ Çelik Dayanımı............................ 116.25 kN

➡ Beton Kopma Dayanımı............. $\boxed{\text{63 kN}}$

➡ Betondan Sıyrılma Dayanımı...... 81.76 kN

ϕN_n = 63 kN

KESME DAYANIMININ BELİRLENMESİ

- Çelik Dayanımı

$V_{sa} = n0.6A_{se,V}f_{uta}$ Ø=0.65 , ***Sünek çelik davranışı***

f_{uta}= 500 Mpa

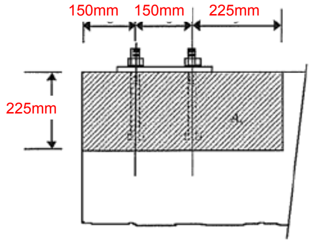
$A_{se,N}$= 155 mm^2

ØV_{sa}=0.65 *2*0.6*155*500= 60450 N = $\boxed{60.45 \text{ kN}}$

- Beton Dayanımı

$$V_{cbg} = \frac{A_{Vc}}{A_{Vco}}\Psi_{ec,V}\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_b$$

Ø=0.70 , Ek donatı düzenlemesi yoktur.



A_{Vco}= 4.5c_{a1}^2=4.5*150^2=101250mm^2

A_{Vc}= 525*225=118125mm^2

★ $\Psi_{ec,V}$=1 (Eksantrisite yoktur)

★ $\Psi_{ed,V}$ 'nin belirlenmesi:

C_{a2} =150mm < 1.5c_{a1} =225mm

$\Psi_{ed,V} = 0.7 + 0.3\dfrac{c_{a2}}{1.5c_{a1}}$ =0.90

★ $\Psi_{c,V}$=1 (beton kenar ve köşeleri çatlak oluşması muhtemel bölgelerdir)

★ $V_b = 0.6\left(\dfrac{\ell_e}{d_a}\right)^{0.2}\sqrt{d_a}\lambda\sqrt{f'_c}\,(c_{a1})^{1.5}$

$\ell_e \le 8d_a$ Şartı sağlanmalıdır.

l_e=8d_a=8 * 16=128mm < h_{ef} =200mm

l_e=128mm

V_b=0.60 * (128/16)$^{0.2}$* SQRT(16) * SQRT(25) * 150$^{1.5}$ =33415 N= 33.42 kN

★★★ ØV_{cbg}=0.70 * (118125/101250)* 1 * 0.90 * 1 * 33.42=$\boxed{24.56 \text{ kN}}$

- Kaldıraç Etkisi Dayanımı

$$V_{cpg} = k_{cp} N_{cbg}$$

Ø=0.70 , Ek donatı düzenlemesi yoktur.

k_{cp} = 2.0

$ØV_{cpg}$=0.70 * 2 * 90 = $\boxed{\text{126 kN}}$

Çelik Dayanımı............................ 60.45 kN

Beton Kopma Dayanımı............. $\boxed{\text{24.6 kN}}$

Kaldıraç Etkisi Dayanımı............ 126 kN

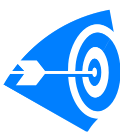
ϕV_n = 24.6 kN

ÇEKME VE KESME ETKİLEŞİMİ

V_{ua}=11 kN >0.2 $ØV_n$=5 kN → tam çekmeye izin verilmez.

N_{ua}=45 kN >0.2 $ØN_n$=12.6 kN → tam kesmeye izin verilmez.

$$\frac{N_{ua}}{\phi N_n} + \frac{V_{ua}}{\phi V_n} \leq 1.2$$



45/63 + 11/24.6 =1.16 < 1.2

ANKRAJ KAPASİTESİ YETERLİDİR...

TEŞEKKÜRLER!

SORULAR ??

CODE

D.3.1.1 — Anchor group effects shall be considered wherever two or more anchors have spacing less than the critical spacing as follows:

Failure mode under investigation	Critical spacing
Concrete breakout in tension	$3h_{ef}$
Bond strength in tension	$2c_{Na}$
Concrete breakout in shear	$3c_{a1}$

Only those anchors susceptible to the particular failure mode under investigation shall be included in the group.

D.3.2 — The design strength of anchors shall equal or exceed the largest required strength calculated from the applicable load combinations in 9.2 or C.9.2.

D.3.3 — Seismic design requirements

D.3.3.1 — Anchors in structures assigned to Seismic Design Category C, D, E, or F shall satisfy the additional requirements of D.3.3.2 through D.3.3.7.

COMMENTARY

stiff. The forces in the anchors are considered to be proportional to the external load and its distance from the neutral axis of the anchor group.

If anchor strength is governed by ductile yielding of the anchor steel, significant redistribution of anchor forces can occur. In this case, an analysis based on the theory of elasticity will be conservative. References D.4 to D.6 discuss nonlinear analysis, using theory of plasticity, for the determination of the capacities of ductile anchor groups.

RD.3.3 — Seismic design requirements

Unless D.3.4.1 or D.3.3.5.1 apply, all anchors in structures assigned to Seismic Design Categories C, D, E, or F are required to satisfy the additional requirements of D.3.3.1 through D.3.3.7 regardless of whether earthquake loads are included in the controlling load combination for the anchor design. In addition, all post-installed anchors in structures assigned to Seismic Design Categories C, D, E, or F must meet the requirements of ACI 355.2 or ACI 355.4M for prequalification of anchors to resist earthquake loads. Ideally, for tension loadings, anchor strength should be governed by yielding of the ductile steel element of the anchor. If the anchor cannot meet the specified ductility requirements of D.3.3.4.3(a), then the attachment should be either designed to yield if it is structural or light gauge steel, or designed to crush if it is wood. If ductility requirements of D.3.3.4.3(a) are satisfied, then any attachments to the anchor should be designed not to yield. In designing attachments using yield mechanisms to provide adequate ductility, as permitted by D.3.3.4.3(b) and D.3.3.5.3(a), the ratio of specified yield strength to expected strength for the material of the attachment should be considered in determining the design force. The value used for the expected strength should consider both material overstrength and strain-hardening effects. For example, the material in a connection element could yield and, due to an increase in its strength with strain hardening, cause a secondary failure of a sub-element or place extra force or deformation demands on the anchors. For a structural steel attachment, if only the specified yield strength of the steel is known, the expected strength should be taken as about 1.5 times the specified yield strength. If the actual yield strength of the steel is known, the expected strength should be taken as about 1.25 times the actual yield strength.

Under seismic conditions, the direction of shear may not be predictable. The full shear force should be assumed in any direction for a safe design.

D

CODE

D.3.3.2 — The provisions of Appendix D do not apply to the design of anchors in plastic hinge zones of concrete structures under earthquake forces.

D.3.3.3 — Post-installed anchors shall be qualified for earthquake loading in accordance with ACI 355.2 or ACI 355.4M. The pullout strength N_p and steel strength in shear V_{sa} of expansion and undercut anchors shall be based on the results of the ACI 355.2 Simulated Seismic Tests. For adhesive anchors, the steel strength in shear V_{sa} and the characteristic bond stresses τ_{uncr} and τ_{cr} shall be based on results of the ACI 355.4M Simulated Seismic Tests.

D.3.3.4 — Requirements for tensile loading

D.3.3.4.1 — Where the tensile component of the strength-level earthquake force applied to a single anchor or group of anchors is equal to or less than 20 percent of the total factored anchor tensile force associated with the same load combination, it shall be permitted to design a single anchor or group of anchors to satisfy D.5 and the tensile strength requirements of D.4.1.1.

D.3.3.4.2 — Where the tensile component of the strength-level earthquake force applied to anchors exceeds 20 percent of the total factored anchor tensile force associated with the same load combination, anchors and their attachments shall be designed in accordance with D.3.3.4.3. The anchor design tensile strength shall be determined in accordance with D.3.3.4.4.

COMMENTARY

RD.3.3.2 — The design provisions in Appendix D do not apply for anchors in plastic hinge zones. The possible higher levels of cracking and spalling in plastic hinge zones are beyond the the conditions for which the nominal concrete-governed strength values in Appendix D are applicable. Plastic hinge zones are considered to extend a distance equal to twice the member depth from any column or beam face, and also include any other sections in walls, frames, and slabs where yielding of reinforcement is likely to occur as a result of lateral displacements.

Where anchors must be located in plastic hinge regions, they should be detailed so that the anchor forces are transferred directly to anchor reinforcement that is specifically designed to carry the anchor forces into the body of the member beyond the anchorage region. Configurations that rely on concrete tensile strength should not be used.

RD.3.3.3 — Anchors that are not suitable for use in cracked concrete should not be used to resist earthquake loads. Qualification of post-installed anchors for use in cracked concrete is an integral part of the qualification for resisting earthquake loads in ACI 355.2 and ACI 355.4M. The design values obtained from the Simulated Seismic Tests of ACI 355.2 and ACI 355.4M are expected to be less than those for static load applications.

RD.3.3.4 — Requirements for tensile loading

RD.3.3.4.1 — The requirements of D.3.3.4.3 need not apply where the applied earthquake tensile force is a small fraction of the total factored tension force.

RD.3.3.4.2 — If the ductile steel element is ASTM A36M or ASTM A307 steel, the f_{uta}/f_{ya} value is typically about 1.5 and the anchor can stretch considerably before rupturing at the threads. For other steels, calculations may need to be made to ensure that a similar behavior can occur. RD.5.1.2 provides additional information on the steel properties of anchors. Provision of upset threaded ends, whereby the threaded end of the rod is enlarged to compensate for the area reduction associated with threading, can ensure that yielding occurs over the stretch length regardless of the ratio of the yield to ultimate strength of the anchor.

CODE

D.3.3.4.3 — Anchors and their attachments shall satisfy one of options (a) through (d):

(a) For single anchors, the concrete-governed strength shall be greater than the steel strength of the anchor. For anchor groups, the ratio of the tensile load on the most highly stressed anchor to the steel strength of that anchor shall be equal to or greater than the ratio of the tensile load on tension-loaded anchors to the concrete-governed strength of those anchors. In each case:

1. The steel strength shall be taken as 1.2 times the nominal steel strength of the anchor.

2. The concrete-governed strength shall be taken as the nominal strength considering pullout, side-face blowout, concrete breakout, and bond strength as applicable. For consideration of pullout in groups, the ratio shall be calculated for the most highly stressed anchor.

In addition, the following shall be satisfied:

3. Anchors shall transmit tensile loads via a ductile steel element with a stretch length of at least eight anchor diameters unless otherwise determined by analysis.

4. Where anchors are subject to load reversals, the anchor shall be protected against buckling.

5. Where connections are threaded and the ductile steel elements are not threaded over their entire length, the ratio of f_{uta}/f_{ya} shall not be less than 1.3 unless the threaded portions are upset. The upset portions shall not be included in the stretch length.

6. Deformed reinforcing bars used as ductile steel elements to resist earthquake effects shall be limited to ASTM A615M Grades 280 and 420 satisfying the requirements of 21.1.5.2(a) and (b) or ASTM A706M Grade 420.

(b) The anchor or group of anchors shall be designed for the maximum tension that can be transmitted to the anchor or group of anchors based on the development of a ductile yield mechanism in the attachment in flexure, shear, or bearing, or a combination of those conditions, and considering both material overstrength and strain hardening effects for the attachment. The anchor design tensile strength shall be calculated from D.3.3.4.4.

(c) The anchor or group of anchors shall be designed for the maximum tension that can be transmitted to the anchors by a non-yielding attachment. The anchor design tensile strength shall be calculated from D.3.3.4.4.

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RD.3.3.4.3 — Four options are provided for determining the required anchor or attachment strength to protect against nonductile tension failure:

In option (a), anchor ductility requirements are imposed and the required anchor strength is that determined using strength-level earthquake forces acting on the structure. Research[D.7,D.8] has shown that if the steel of the anchor yields before the concrete anchorage fails, no reduction in the anchor tensile strength is needed for earthquake loadings. Ductile steel anchors should satisfy the definition for ductile steel elements in D.1. To facilitate comparison between steel strength, which is based on the most highly-stressed anchor, and concrete strength based on group behavior, the design is performed on the basis of the ratio of applied load to strength for the steel and concrete, respectively.

For some structures, anchors provide the best locations for energy dissipation in the nonlinear range of response. The stretch length of the anchor affects the lateral displacement capacity of the structure and therefore that length needs to be sufficient such that the displacement associated with the design-basis earthquake can be achieved.[D.9] Observations from earthquakes indicate that the provision of a stretch length of eight anchor diameters results in good structural performance. Where the required stretch length is calculated, the relative stiffness of the connected elements needs to be considered. When an anchor is subject to load reversals, and its yielding length outside the concrete exceeds six anchor diameters, buckling of the anchor in compression is likely. Buckling can be restrained by placing the anchor in a tube. However, care must be taken that the tube does not share in resisting the tensile load assumed to act on the anchor. For anchor bolts that are not threaded over their length, it is important to ensure that yielding occur over the unthreaded portion of the bolt within the stretch length prior to failure in the threads. This is accomplished by maintaining sufficient margin between the specified yield and ultimate strengths of the bolt. It should be noted that the available stretch length may be adversely influenced by construction techniques (for example, the addition of leveling nuts to the examples shown in Fig. RD.1.3).

In option (b), the anchor is designed for the tension force associated with the expected strength of the metal or similar material of the attachment. For option (b), as discussed in RD.3.3, care must be taken in design to consider the consequences of potential differences between the specified yield strength and the expected strength of the attachment. An example is 21.4.3 for the design of connections of intermediate precast walls where a connection not designed to yield should develop at least $1.5S_y$, where S_y is the nominal strength of the yielding element based on its specified yield strength. Similarly, steel design manuals require structural steel connections that are designated nonyielding and part of the seismic load path to have design strengths that exceed a

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(d) The anchor or group of anchors shall be designed for the maximum tension obtained from design load combinations that include E, with E increased by Ω_o. The anchor design tensile strength shall satisfy the tensile strength requirements of D.4.1.1.

D.3.3.4.4 — The anchor design tensile strength for resisting earthquake forces shall be determined from consideration of (a) through (e) for the failure modes given in Table D.4.1.1 assuming the concrete is cracked unless it can be demonstrated that the concrete remains uncracked:

(a) ϕN_{sa} for a single anchor, or for the most highly stressed individual anchor in a group of anchors;

(b) $0.75\phi N_{cb}$ or $0.75\phi N_{cbg}$, except that N_{cb} or N_{cbg} need not be calculated where anchor reinforcement satisfying D.5.2.9 is provided;

(c) $0.75\phi N_{pn}$ for a single anchor, or for the most highly stressed individual anchor in a group of anchors;

(d) $0.75\phi N_{sb}$ or $0.75\phi N_{sbg}$; and

(e) $0.75\phi N_a$ or $0.75\phi N_{ag}$

where ϕ is in accordance with D.4.3 or D.4.4.

D.3.3.4.5 — Where anchor reinforcement is provided in accordance with D.5.2.9, no reduction in design tensile strength beyond that specified in D.5.2.9 shall be required.

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multiple of the nominal strength. That multiple depends on a factor relating the likely actual to specified yield strength of the material and an additional factor exceeding unity to account for material strain hardening. For attachments of cold-formed steel or wood, similar principles should be used for determining the expected strength of the attachment in order to determine the required strength of the anchorage.

Additional guidance on the use of options (a) through (d) is provided in the *2009 NEHRP Provisions*.[D.9] The design of anchors in accordance with option (a) should be used only where the anchor yield behavior is well defined and where the interaction of the yielding anchor with other elements in the load path has been adequately addressed. For the design of anchors per option (b), the force associated with yield of a steel attachment, such as an angle, baseplate, or web tab, should be the expected strength, rather than the specified yield strength of the steel. Option (c) may apply to a variety of special cases, such as the design of sill bolts where the crushing of the wood limits the force that can be transferred to the bolt, or where the provisions of AISC 341, Seismic Provisions for Structural Steel Buildings,[D.10] specify loads based on member strengths.

RD.3.3.4.4 — The reduced anchor nominal tensile strengths associated with concrete failure modes is to account for increased cracking and spalling in the concrete resulting from seismic actions. Because seismic design generally assumes that all or portions of the structure are loaded beyond yield, it is likely that the concrete is cracked throughout for the purpose of determining the anchor strength. In locations where it can be demonstrated that the concrete does not crack, uncracked concrete may be assumed for determining the anchor strength as governed by concrete failure modes.

RD.3.3.4.5 — Where anchor reinforcement as defined in D.5.2.9 and D.6.2.9 is used, with the properties as defined in 21.1.5.2, no separation of the potential breakout prism from the substrate is likely to occur provided the anchor reinforcement is designed for a load greater than the concrete breakout strength.

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D.3.3.5 — Requirements for shear loading

RD.3.3.5 — Requirements for shear loading

Where the shear component of the earthquake force applied to the anchor exceeds 20 percent of the total anchor shear force, three options are recognized for determining the required shear strength to protect the anchor or group of anchors against premature shear failure. There is no option corresponding to option (a) of D.3.3.4.3 because the cross section of the steel element of the anchor cannot be configured so that steel failure in shear provides any meaningful degree of ductility.

Design of the anchor or group of anchors for the strength associated with force-limiting mechanisms under option (b), such as the bearing strength at holes in a steel attachment or the combined crushing and bearing strength for wood members may be particularly relevant. Tests on typical anchor bolt connections for wood framed shear walls[D.11] showed that wood components attached to concrete with minimum edge distances exhibited ductile behavior. Wood "yield" (crushing) was the first limiting state and resulted in nail slippage in shear. Nail slippage combined with bolt bending provided the required ductility and toughness for the shear walls and limited the loads acting on the bolts. Procedures for defining bearing and shear limit states for connections to cold-formed steel are described in AISI S100-07[D.12] and examples of strength calculations are provided in the AISI "Cold-Formed Steel Design Manual."[D.13] In such cases, consideration should be given to whether exceedance of the bearing strength may lead to tearing and an unacceptable loss of connectivity. Where anchors are located far from edges it may not be possible to design such that anchor reinforcement controls the anchor strength. In such cases, anchors should be designed for overstrength in accordance with option (c).

D.3.3.5.1 — Where the shear component of the strength-level earthquake force applied to the anchor or group of anchors is equal to or less than 20 percent of the total factored anchor shear force associated with the same load combination, it shall be permitted to design the anchor or group of anchors to satisfy D.6 and the shear strength requirements of D.4.1.1.

RD.3.3.5.1 — The requirements of D.3.3.5.3 need not apply where the applied earthquake shear force is a small fraction of the total factored shear force.

D.3.3.5.2 — Where the shear component of the strength-level earthquake force applied to anchors exceeds 20 percent of the total factored anchor shear force associated with the same load combination, anchors and their attachments shall be designed in accordance with D.3.3.5.3. The anchor design shear strength for resisting earthquake forces shall be determined in accordance with D.6.

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D.3.3.5.3 — Anchors and their attachments shall be designed using one of options (a) through (c):

(a) The anchor or group of anchors shall be designed for the maximum shear that can be transmitted to the anchor or group of anchors based on the development of a ductile yield mechanism in the attachment in flexure, shear, or bearing, or a combination of those conditions, and considering both material overstrength and strain hardening effects in the attachment.

(b) The anchor or group of anchors shall be designed for the maximum shear that can be transmitted to the anchors by a non-yielding attachment.

(c) The anchor or group of anchors shall be designed for the maximum shear obtained from design load combinations that include E, with E increased by Ω_o. The anchor design shear strength shall satisfy the shear strength requirements of D.4.1.1.

D.3.3.5.4 — Where anchor reinforcement is provided in accordance with D.6.2.9, no reduction in design shear strength beyond that specified in D.6.2.9 shall be required.

D.3.3.6 — Single anchors or groups of anchors that are subjected to both tension and shear forces shall be designed to satisfy the requirements of D.7, with the anchor design tensile strength calculated from D.3.3.4.4.

D.3.3.7 — Anchor reinforcement used in structures assigned to Seismic Design Category C, D, E, or F shall be deformed reinforcement and shall be limited to ASTM A615M Grades 280 and 420 satisfying the requirements of 21.1.5.2(a) and (b) or ASTM A706M Grade 420.

D.3.4 — Adhesive anchors installed horizontally or upwardly inclined shall be qualified in accordance with ACI 355.4M requirements for sensitivity to installation direction.

D.3.5 — For adhesive anchors subjected to sustained tension loading, D.4.1.2 shall be satisfied. For groups of adhesive anchors, Eq. (D-1) shall be satisfied for the anchor that resists the highest sustained tension load. Installer certification and inspection requirements for horizontal and upwardly inclined adhesive anchors subjected to sustained tension loading shall be in accordance with D.9.2.2 through D.9.2.4.

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RD.3.4 — ACI 355.4M includes optional tests to confirm the suitability of adhesive anchors for horizontal and upwardly inclined installations.

RD.3.5 — For adhesive anchors subjected to sustained tension loading, an additional calculation for the sustained portion of the factored load for a reduced bond resistance is required to account for possible bond strength reductions under sustained load. The resistance of adhesive anchors to sustained tension load is particularly dependent on correct installation, including hole cleaning, adhesive metering and mixing, and prevention of voids in the adhesive bond line (annular gap). In addition, care should be taken in the selection of the correct adhesive and bond strength for the expected conditions on-site such as the concrete condition during

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D.3.6 — Modification factor λ_a for lightweight concrete shall be taken as:

Cast-in and undercut anchor concrete failure ... **1.0λ**

Expansion and adhesive anchor concrete failure... **0.8λ**

Adhesive anchor bond failure per Eq. (D-22) **0.6λ**

where λ is determined in accordance with 8.6.1. It shall be permitted to use an alternate value of λ_a where tests have been performed and evaluated in accordance with ACI 355.2 or ACI 355.4M.

D.3.7 — The values of f_c' used for calculation purposes in this appendix shall not exceed 70 MPa for cast-in anchors, and 55 MPa for post-installed anchors. Testing is required for post-installed anchors when used in concrete with f_c' greater than 55 MPa.

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installation (dry or saturated, cold or hot), the drilling method used (rotary impact drill/rock drill or core drill), and anticipated in-service temperature variations in the concrete. Installer certification and inspection requirements associated with the use of adhesive anchors for horizontal and upwardly inclined installations to resist sustained tension loads are addressed in D.9.2.2 through D.9.2.4.

Adhesive anchors are particularly sensitive to installation direction and loading type. Adhesive anchors installed overhead that resist sustained tension loads are of concern since previous applications of this type have led to failures. Other anchor types may be more appropriate for such cases. Where adhesive anchors are used in overhead applications subjected to sustained tension loading, it is essential to meet test requirements of ACI 355.4M for sensitivity to installation direction, use certified installers, and require special inspection.

RD.3.6 — The number of tests available to establish the strength of anchors in lightweight concrete is limited. Lightweight concrete tests of cast-in headed studs indicate that the present reduction factor λ adequately captures the influence of lightweight concrete.[D.14,D.15] Anchor manufacturer data developed for evaluation reports on both post-installed expansion and adhesive anchors indicate that a reduced λ is needed to provide the necessary safety factor for the respective design strength. ACI 355.2 and ACI 355.4M provide procedures whereby a specific value of λ_a can be used based on testing, assuming the lightweight concrete is similar to the reference test material.

RD.3.7 — A limited number of tests of cast-in and post-installed anchors in high-strength concrete[D.16] indicate that the design procedures contained in this appendix become unconservative, particularly for cast-in anchors in concrete with compressive strengths in the range of 75 to 85 MPa. Until further tests are available, an upper limit on f_c' of 70 MPa has been imposed in the design of cast-in anchors. This limitation is consistent with Chapters 11 and 12. ACI 355.2 and ACI 355.4M do not require testing of post-installed anchors in concrete with f_c' greater than 55 MPa. Some post-installed expansion anchors may have difficulty expanding in very high-strength concretes and the bond strength of adhesive anchors may be negatively affected by very high-strength concrete. Therefore, f_c' is limited to 55 MPa in the design of post-installed anchors unless testing is performed.