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Parking structures are subjected to volume change stresses, which may cause distress in framing elements. Expansion joints are generally introduced in the structures to alleviate the impact of volume change effects. This paper addresses the threshold question of whether an expansion joint is needed in a parking facility and provides performance-based guidelines to assess the need for an expansion joint.

KEYWORDS

Creep; design; expansion joint; parking structures; prestress; restraint; shrinkage; temperature; volume change.

INTRODUCTION

Most parking structures are built using concrete. The structures are open and unheated. As such, they are subjected to creep, shrinkage, and temperature (C-S-T) effects. Further, in the case of post-tensioned structures, floor shortening caused by precompression adds to the C-S-T effects in causing structural movement in parking structures. The four factors are jointly known as volume change (VC) effects.[1] The restraint to volume changes induces stresses that can cause cracks, leaks, and premature deterioration in concrete structures. To design for the VC effects, ACI 318 requires that design be based on a "realistic assessment" of such effects occurring in service[2]; however, concrete is a complex nonlinear material. Unlike steel that undergoes neither creep nor shrinkage and has a well-defined coefficient of thermal expansion, concrete offers challenges in realistically assessing its VC effects. The VC deformations accumulate over the building length, as shown in Fig. 1. To limit the stress buildup, expansion joints are introduced by providing an opening or a gap between adjacent structural segments starting from the ground level up to the roof, as shown in Fig. 2. The term "expansion joint" is a misnomer because the joints are introduced primarily to allow shortening, and not expansion, of the structure. The expansion joints are expensive to install and maintain and, if not maintained, they present a potential hazard to pedestrians who may trip over them and to motorists whose vehicles may bottom out. Therefore, it is desirable to minimize expansion joints in parking structures.

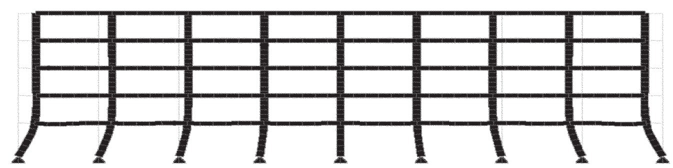

Fig. 1—Deformed shape of a moment subjected to VC effects.

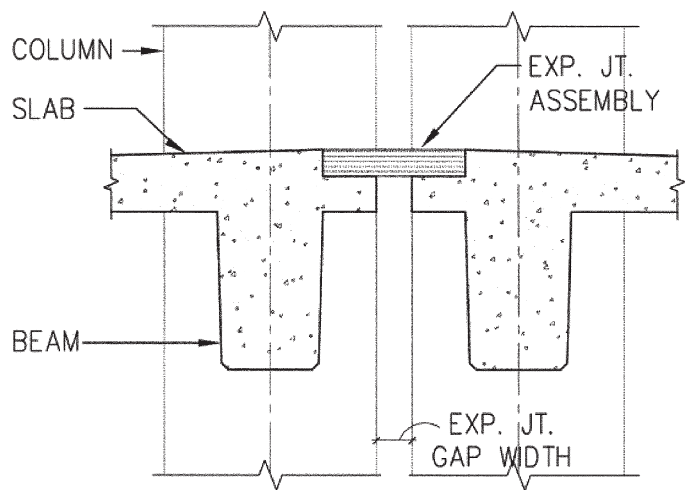

Fig. 2—Double-column expansion joint in post-tensioned parking structure.

This paper presents the state-of-the-art review of prescriptive requirements to design expansion joints in parking structures, along with recent insight gained in

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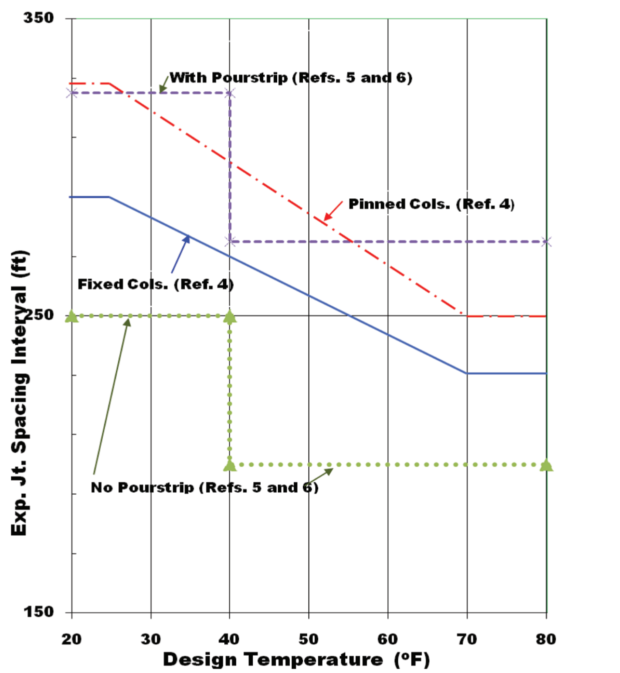

Fig. 3—Expansion joint spacing guidelines per References 4 through 6 (Note: 1 ft = 0.31 m and 1°F of design temperature [temperature change] = 0.55°C.)

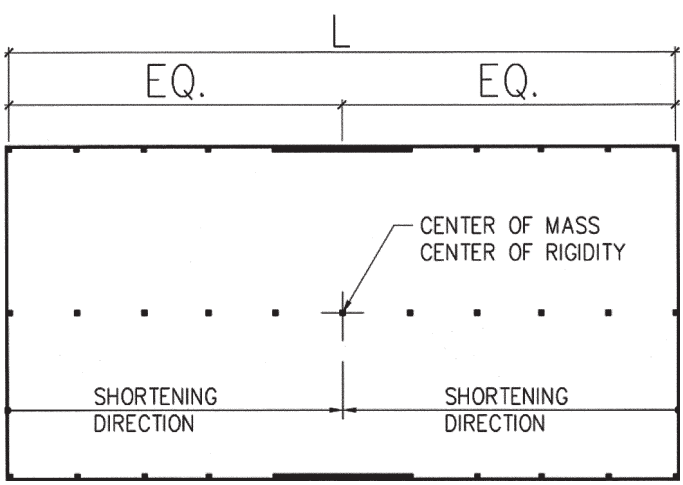

Fig. 4—LFRS layout to maximize expansion joint spacing (adapted from Reference 5).

understanding the performance of parking structures under VC effects. Next, major factors influencing the need for expansion joints are discussed. Design guidelines are presented at the end.[*]

HISTORICAL BACKGROUND

A threshold issue in expansion joint design is to determine the locations or spacing intervals at which the joints should be installed. This issue was addressed 35-plus years ago by the Federal Construction Council (FCC), which developed the first guidelines based on measurements recorded in 1943 in nine buildings.[4] For rectangular buildings, the FCC required the expansion joint spacing criteria to consider two factors: design temperature and column base fixity, as shown in Fig. 3. Recognizing the difference in behavior of precast and post-tensioned concrete structural systems, Chrest et al.[5] proposed that expansion joints in post-tensioned parking structures should be spaced at 200 ft (61 m) maximum; however, if the floor diaphragm had a pourstrip, the spacing could be increased to 275 ft (84 m).[5] Subsequently, the Post-Tensioning Institute (PTI) published its expansion joint guidelines. The PTI guidelines[6] are identical to the Chrest et al.[5] guidelines, except that PTI increased the expansion joint spacing limits by 50 ft (15 m) in each case, with a caveat that the recommended limits were meant for locations where temperature changes were "not significant" and that they should be modified for locations with "significant" temperature changes. PTI did not define what constitutes a "significant" temperature change; however, it emphasized that the guidelines were based on ideal structural framing layout so that stiff elements such as shear walls are located at or near the center of rigidity of the structure where little movement was expected. Figure 4 illustrates an ideal layout where the center of mass and center of rigidity of the facility coincide. The Chrest et al.[5] and PTI[6] recommendations were complementary to each other because Chrest et al.[5] based their recommendation on parking structures located in the Midwest where temperature differentials are significant and durability is a primary concern, whereas PTI aimed its recommendation for the southwestern part of the U.S. with mild or "nonsignificant" temperatures. Assuming a design temperature (temperature change) of 40°F (22°C) is the line between the significant and nonsignificant temperatures, the PTI[6] and Chrest et al.[5] recommendations are shown in Fig. 3. Separately, the Precast/Prestressed Concrete Institute (PCI) recommended considering connection deformability, frame stiffness, location of lateral force resisting system (LFRS), and weather exposure conditions in assessing the need for an expansion joint.[7] However, PCI did not

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[*]The design guidelines to size the gap at an expansion join (Fig. 1) were published in Reference 3 and are not repeated in this paper.

provide any design criteria for the factors affecting the need for an expansion joint in a parking structure.

The purpose of providing an expansion joint is to reduce the VC stress build-up to a tolerable level, and not to eliminate the build-up altogether.[2] The ACI 318 commentary states that, where LFRS provides "significant restraint" to shrinkage and temperature movements, it may be necessary to increase the amount of slab reinforcement required to control cracking. ACI 318 does not define what "significant restraint" is. Therefore, the expansion joint spacing criterion remains an unsettled issue; however, establishing a rational expansion joint spacing criterion is important because omitting an expansion joint where it is needed creates a risk of structural distress, causing unnecessary repair costs. On the other hand, installing an expansion joint where it is not needed increases initial construction cost and adds to the maintenance costs of the facility. This paper presents guidelines to optimize the expansion joint spacing and thus enhance structural performance. The guidelines are based on this author's experience in design of parking structures and in resolving matters related to expansion joints.

DESIGN FACTORS

The need to install an expansion joint depends on several factors including design temperature, structural system type, stiffness, integrity, and framing layout. It is well known that the volume changes increase with the distance from the center of rigidity and reach their maximum at the perimeter (Fig. 1). Because shortening has more pronounced impact on cracking than its expansion counterpart, more emphasis is given to the shortening aspect; however, shortening in a constructed facility does take place in a free or unrestrained manner but is, to a varying degree, restrained by LFRS. An understanding of factors causing shortening on one hand, and restraints to shortening on the other, is essential to assess the need for the expansion joint spacing. The significant factors are discussed as follows.

Design temperature

It has been recognized that seasonal temperature changes from anticipated high temperature (T_{max}) to low temperature (T_{min}) in a locality are the principal cause of shortening. Generally, it takes at least a year to construct a parking structure and, therefore, various structural elements are installed at different temperatures. To simplify determination of design temperature for a structure, the mean construction season temperature (T_c) is generally used.[3] As construction is carried out generally in above-freezing temperatures, the temperature T_c is invariably above 32°F (0°C). What temperature values should be used for T_{max}, T_{min}, and T_c for a location is an important design step. The FCC[4] defined T_{max} as the temperature exceeded, on average, only 1% of the time during the summer months. Similarly, T_{min} is defined as the temperature that equals or exceeds, on average, 99% of the time during the winter months in the locality of the building. The list of temperature variations for cities across the U.S. is given in Table 1. Because thermal shortening is additive to creep and shrinkage effects and manifests in structural distress, the design temperature Δ_T is defined as

$$\Delta_T = T_c - T_{min} \tag{1}$$

As shown in Table 1, the design temperature Δ_T varies from 16°F (9°C) for Honolulu, HI, to 103°F (57°C) for Fairbanks, AK, with cities in the 48 contiguous states falling between the two.

Structural system

Three types of structural systems used in parking structures are post-tensioned, pretopped double-T, and field-topped double-T precast concrete systems. The structural system type selected has considerable impact on thermal movements. The post-tensioned structures exhibit more thermal movement than their precast counterpart.[1] To alleviate the stress build-up in the post-tensioned diaphragms and to minimize associated cracking, control strips are introduced during construction. The control strips, also called closure strips, delay strips, or pour-strips, help increase flexibility and thus permit initial creep and shrinkage to dissipate. Once the pour strip concrete attains its design strength, the pour strip loses its effectiveness and both sides of the diaphragm act as one unit during the service life of the structure.

Structural stiffness

It is well known that structures do not move freely under VC effects, as their movement is inhibited by structural stiffness. An increase in the LFRS stiffness increases restraint and axial tension in the diaphragm. As a result, slab and beams located near the LFRS's center of rigidity are subjected to the maximum axial stress while its exterior columns are subjected to the maximum bending moment and shear forces.[1] If the LFRS is too stiff to flex, the VC stresses would cause significant extensive cracking in slab and other elements. In this respect, the published guidelines may be unconservative since they do not include LFRS stiffness in expansion joint need assessment.

Table 1—Temperature variation in the United States[4]

Location	T_{max}	T_{min}	T_c
Alabama			
Birmingham	97	63	19
Huntsville	97	61	13
Mobile	96	68	28
Montgomery	98	66	22
Alaska			
Anchorage	73	51	–25
Barrow	58	38	–45
Fairbanks	82	50	–53
Juneau	75	48	–7
Nome	66	45	–32
Arizona			
Flagstaff	84	58	0
Phoenix	108	70	31
Prescott	96	64	15
Tucson	105	67	29
Winslow	97	67	9
Yuma	111	72	37
Arkansas			
Ft. Smith	101	65	15
Little Rock	99	65	19
Texarkana	99	65	22
California			
Bakersfield	103	65	31
Burbank	97	64	36
Eureka/Arcata	67	52	32
Fresno	101	63	28
Long Beach	87	63	41
Los Angeles	94	62	41
Oakland	85	57	35
Sacramento	100	60	30
San Diego	86	62	42
San Francisco	83	56	35
Santa Maria	57	57	32

Location	T_{max}	T_{min}	T_c
Colorado			
Alamosa	84	60	–17
Colorado Springs	90	61	–1
Denver	92	62	–2
Grand Junction	96	64	8
Pueblo	96	64	–5
Connecticut			
Bridgeport	90	60	4
Hartford	90	61	1
New Haven	88	59	5
Delaware			
Wilmington	93	62	12
Florida			
Daytona Beach	94	70	32
Ft. Myers	94	74	38
Jacksonville	96	68	29
Key West	90	77	55
Lakeland	95	72	35
Miami	92	75	44
Miami Beach	91	75	45
Orlando	96	72	33
Pensacola	92	68	29
Tallahassee	96	68	25
Tampa	92	72	36
West Palm Beach	92	75	40
Georgia			
Athens	96	61	17
Atlanta	95	62	18
Augusta	98	64	20
Columbus	98	65	23
Macon	98	65	23
Rome	97	62	16
Savannah/Travis	96	67	24
Hawaii			
Hilo	85	73	59
Honolulu	87	76	60
Idaho			
Boise	96	61	4
Idaho Falls	91	61	–12
Lewiston	98	60	6
Pocatello	94	60	–8

Location	T_{max}	T_{min}	T_c
Illinois			
Chicago	95	60	–3
Moline	94	63	–7
Peoria	94	61	–2
Rockford	92	62	–7
Springfield	95	62	–1
Indiana			
Evansville	96	65	6
Fort Wayne	93	62	0
Indianapolis	93	63	0
South Bend	92	61	–2
Iowa			
Burlington	95	64	–4
Des Moines	95	64	–7
Dubuque	62	63	–11
Sioux City	96	64	–10
Waterloo	91	63	–12
Kansas			
Dodge City	99	64	3
Goodland	99	65	–2
Topeka	99	69	3
Wichita	102	68	5
Kentucky			
Covington	93	63	3
Lexington	94	63	6
Louisville	96	64	8
Louisiana			
Baton Rouge	96	68	25
Lake Charles	95	68	29
New Orleans	93	69	32
Shreveport	99	66	22
Maine			
Caribou	85	56	–18
Portland	88	58	–5
Maryland			
Baltimore	94	63	12
Frederick	94	63	7
Massachusetts			
Boston	91	58	6

Location	T_{max}	T_{min}	T_c
Massachussetts, cont.			
Pittsfield	86	58	–5
Worcester	89	58	–3
Michigan			
Alpena	87	57	–5
Detroit-Metropolitan	92	58	4
Escanaba	82	55	–7
Flint	89	60	–1
Grand Rapids	91	62	2
Lansing	89	59	2
Marquette	88	55	–8
Muskegon	87	59	4
Sault Ste. Marie	83	55	–12
Minnesota			
Duluth	85	55	–19
International Falls	86	57	–29
Minneapolis/St. Paul	92	62	–14
Rochester	90	60	–17
St. Cloud	90	60	–20
Mississippi			
Jackson	98	66	21
Meridian	97	65	20
Vicksburg	97	66	23
Missouri			
Columbia	97	65	2
Kansas City	100	65	1
St. Joseph	97	66	–1
St. Louis	98	65	4
Springfield	97	64	5
Montana			
Billings	94	60	–10
Glasgow	96	60	–25
Great Falls	91	58	–20
Havre	91	58	–22
Helena	90	58	–17
Kalispell	88	56	–7
Miles City	97	62	–19
Missoula	92	58	–7
Nebraska			
Grand Island	98	65	–6
Lincoln	100	64	–4

Location	T_{max}	T_{min}	T_c
Nebraska, cont.			
Norfolk	97	64	–11
North Platte	97	64	–6
Omaha	97	64	–5
Scottsbluff	96	62	–8
Nevada			
Elko	94	61	–13
Ely	90	59	–6
Las Vegas	108	66	23
Reno	95	62	2
Winnemucca	97	63	1
New Hampshire			
Concord	91	60	–11
New Jersey			
Atlantic City	91	61	14
Newark	94	62	11
Trenton	92	61	12
New Mexico			
Albuquerque	96	64	14
Raton	92	64	–2
Roswell	101	70	16
New York			
Albany	91	61	–5
Binghampton	91	66	–2
Buffalo	88	59	3
New York	94	59	11
Rochester	91	53	2
Syracuse	90	59	–2
North Carolina			
Asheville	91	60	13
Charlotte	96	60	18
Greensboro	94	64	14
Raleigh/Durham	95	62	16
Wilmington	93	63	23
Winston/Salem	94	63	14
North Dakota			
Bismarck	95	60	–24
Devils Lake	93	58	–23
Fargo	92	59	–22
Minot	91	—	–24

Location	T_{max}	T_{min}	T_c
North Dakota, cont.			
Williston	94	59	–21
Ohio			
Akron/Canton	89	60	1
Cincinnati	94	62	8
Cleveland	91	61	2
Columbus	92	61	2
Dayton	92	61	0
Mansfield	91	61	1
Sandusky	92	60	4
Toledo	92	61	1
Youngstown	89	59	1
Oklahoma			
Oklahoma City	100	64	11
Tulsa	102	65	12
Oregon			
Astoria	79	50	27
Eugene	91	52	22
Medford	98	56	21
Pendleton	97	58	3
Portland	91	52	21
Roseburg	93	54	25
Salem	92	52	21
Pennsylvania			
Allentown	92	61	3
Erie	88	59	7
Harrisburg	92	61	9
Philadelphia	93	63	11
Pittsburgh	90	63	5
Reading	92	61	3
Scranton Wilkes-Barre	89	61	2
Williamsport	91	61	1
Rhode Island			
Providence	89	60	6
South Carolina			
Charleston	95	66	23
Columbia	98	64	20
Florence	96	64	21
Greenville	95	61	19
Spartanburg	95	60	18

Location	T_{max}	T_{min}	T_c
South Dakota			
Huron	97	62	−16
Rapid City	96	61	−9
Sioux Falls	95	62	−14
Tennessee			
Bristol/Tri City	92	63	11
Chattanooga	97	60	15
Knoxville	95	60	13
Memphis	98	62	17
Nashville	97	62	12
Texas			
Abilene	101	65	17
Amarillo	98	66	8
Austin	101	68	25
Brownsville	94	74	36
Corpus Christi	95	71	32
Dallas	101	66	19
El Paso	100	65	21
Fort Worth	102	66	20
Galveston	91	70	32
Loredo AFB	103	74	32
Lubbock	99	67	11
Midland	100	66	19
Port Arthur	94	69	29
San Angelo	101	65	20
San Antonio	99	69	25
Victoria	98	71	28
Waco	101	67	21
Wichita Falls	103	66	15
Utah			
Salt Lake City	97	63	5
Vermont			
Burlington	88	57	−12
Virginia			
Lynchburg	94	62	15
Norfolk	94	60	20
Richmond	96	64	14
Roanoke	94	63	15
Washington, DC			
National Airport	94	63	16

Location	T_{max}	T_{min}	T_c
Washington			
Olympia	85	51	21
Seattle	85	51	20
Spokane	93	58	−2
Walla Walla	98	57	12
Yakima	94	62	6
West Virginia			
Charleston	92	63	9
Huntington	95	63	10
Parkersburg	93	62	8
Wisconsin			
Green Bay	88	59	−12
La Crosse	90	62	−12
Madison	92	61	−9
Milwaukee	90	60	−6
Wyoming			
Casper	92	59	−11
Cheyenne	89	58	−6
Lander	92	58	−16
Sheridan	95	59	−12

It should be pointed out that LFRS stiffness and VC movements requiring flexibility are mutually exclusive or contradictory requirements that need to be reconciled. Though structural stiffness is needed for structural stability under anticipated loading conditions, the structures should be able to flex enough to keep VC stresses in check for proper performance. Therefore, the question becomes to what extent can the structural movements be restrained so that a structure continues to perform well under VC effects. One way to address this issue is to quantify the restraining effect of LFRS stiffness and thus keep it within certain limits. The movement factor concept serves this purpose and is discussed as follows.

Movement factor

The movement factor is as an index that quantifies structural flexibility in parking structures.[1] The movement factor M is defined as the ratio of actual movement and the calculated unrestrained VC movement in a diaphragm and varies from 0 and 1, depending on the LFRS stiffness. For a freely moving diaphragm under volume changes, the M factor is unity. On the other hand, a diaphragm that is fully restrained by rigid elements on both ends has an M

Fig. 5—Temporary hinge ("sand pocket") provided at column base to reduce LFRS stiffness during construction phase.

factor of zero. The lower bound of the *M* factor for post-tensioned structures performing satisfactorily in service has been determined to be 0.8.[1] The *M* factor of 0.8 means that a structure moves 80% of the totally unrestrained movement with the remaining 20% movement inhibited by the structural restraint. The 80% movement level also indicates the maximum degree of restraint post-tensioned structures may tolerate while performing reasonably well. For design purposes, the *M* factor may be defined as a ratio of the first-story VC movement and the corresponding unrestrained VC movement. If an analytical model has an *M* factor of less than 0.8, it is likely that the underlying structure, when constructed, would not perform well. In such a case, the LFRS should be reexamined and measures should be taken to reduce its stiffness.

One approach to reduce the LFRS stiffness is by reducing the stiffness of the first story columns. Another way to deal with the excess stiffness is to introduce vertical pourstrips in columns, as shown in Fig. 5.[8] The vertical pourstrips, also known as sand-pockets, reduce LFRS stiffness, reduce post-tensioning losses, and allow the diaphragm to move relatively freely during the construction phase. Thus, they reduce creep and shrinkage accumulations during the construction phase; however, they offer little relief after concrete is placed in the sand pockets.

Structural framing layout

To optimize the use of expansion joints, parking structures need to have a rectangular footprint with the LFRS centered at or near the center of mass of the diaphragm, as shown in Fig. 4. In cases where it is not practical to optimally place the LFRS, the permissible length between the expansion joints should be adjusted using the guidelines presented as follows.

DESIGN GUIDELINES

In light of the aforementioned, the structures using the expansion joint spacing rules prescribed in Fig. 3 without considering restraint caused by its LFRS may not perform satisfactorily during their life time. On the other hand, several structures are in service with expansion joint spacing that exceeds the limits prescribed in Fig. 3. The structures have been designed, detailed, and built keeping the structural restraints within reasonable limits. The following guidelines are presented to assess expansion joint needs in post-tensioned parking structures:

1. The design temperature, structural system type, stiffness, and framing layout are significant parameters in assessing the need for an expansion joint.

2. The design temperature should be computed using Eq. (1).

3. It is preferable that parking structures have a rectangular footprint with the LFRS centered at or near the center of mass of the diaphragm, as shown in Fig. 4.

4. Use the *M* factor method. The factor *M* can be expressed as:

$$M = \frac{\Delta_1}{\Delta} \tag{2}$$

where Δ_1 is the LFRS movement at the first supported level with the diaphragm under anticipated thermal strain, $e_{th}\Delta T$; Δ is the unrestrained anticipated movement = $e_{th}\Delta TX$; e_{th} is the coefficient of thermal expansion of concrete = 7.5×10^{-6} in./in./°F[1] (13.5×10^{-6} m/m/°C); ΔT is the design temperature; and X is the building length contributing to volume changes at the expansion joint.

5. In determining Δ_1, the structural model is subjected to the anticipated strain. It is suggested that post-tensioned floor elements should be considered uncracked. The reinforced concrete elements such as columns may be considered cracked. The extent of allowable cracking depends on the project requirements; however, the effective moment of inertia of the columns used in analysis should not be less than 50% of the gross moment of inertia

6. A structure having an *M* factor value of 0.9 is considered a moderately rigid structure with a small likelihood of VC cracking. A structure with an *M* factor of less than 0.8 is considered stiff and is likely to experience

cracking and may require additional reinforcement in various elements to control cracking.

7. For structures with an *M* factor of less than 0.8, VC stresses can be reduced using sand pockets. However, the number and locations of sand pockets needed to reduce stiffness to a reasonable level depends on the extent of distress and cracking that can be tolerated during the structure's lifetime.

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