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Parking structures generally have expansion joints to minimize the impact of volume change stresses. Expansion joint separation gaps should be wide enough to allow volume change movements and lateral drifts to avoid structural pounding during a seismic event. This paper presents literature review and demonstrates a design method to optimize sizing of such gaps, which can be used in other concrete buildings as well.

KEYWORDS

Concrete; construction; design; expansion joint; parking; post-tensioning; retrofit; seismic pounding; volume change.

INTRODUCTION

Most parking structures are built using concrete. The structures have large footprints, often exceed 300 ft (90 m) in length and adjoin other buildings and bridges, as shown in Fig. 1. Parking 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 by causing movement in parking structures. The four factors are jointly known as volume change (VC) effects.[1] To minimize the VC-induced force buildup, expansion joints are introduced, dividing a structure into segments. An expansion joint is introduced by providing an opening or a gap between two structural segments starting from the top of the foundation and continuing throughout the height of the structure, as shown in Fig. 2. A commonly used expansion joint in parking structures is one with two closely spaced columns with a gap between them, as shown in Fig. 3. Once a design professional has determined expansion joint locations, the next step is to size the joints or, in other words, determine the separation

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Fig. 1—Post-tensioned parking structure in Mobile, AL. Structure has three expansion joints: (1) between bridge and stair/elevator tower; (2) between main parking structure and stair/elevator tower at front end; and (3) between stair tower and parking area on the back end of structure. (Photo courtesy of Walker Parking Consultants.)

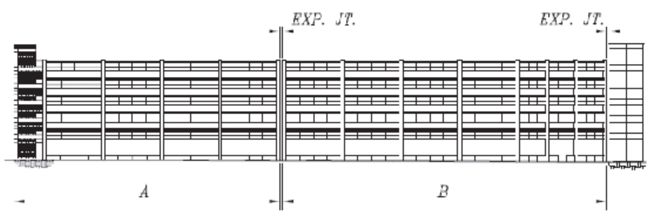

Fig. 2—Elevation of parking structure with two expansion joints. One expansion joint is between portions A and B of structure and other is between portion B and stair/elevator on right. (Architect: Hnedak Bobo Group, Inc.; Engineer: Walker Parking Consultants.)

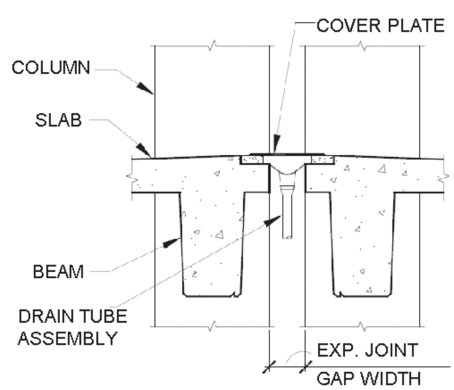

Fig. 3—Double-column expansion joint.

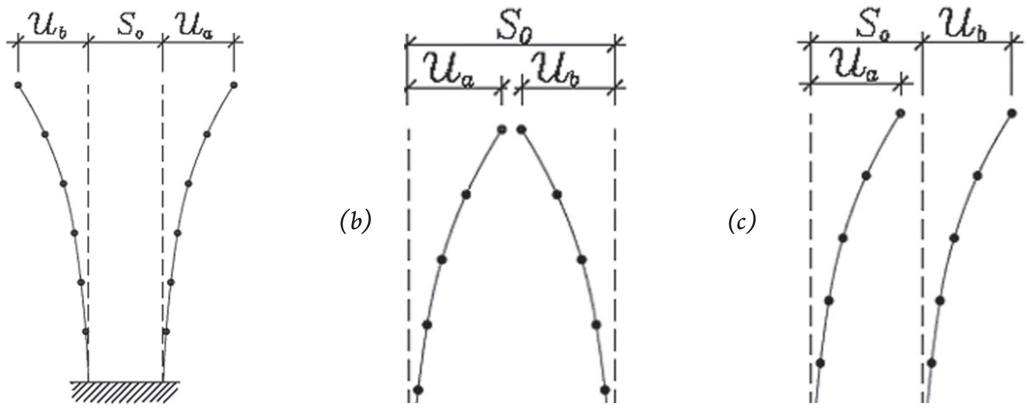

Fig. 4—(a) Out-of-phase outward movement of adjacent buildings; (b) out-of-phase inward movement of adjacent buildings; and (c) in-phase drift of adjacent buildings.

gap width between the adjacent building units, as shown in Fig. 3.

Expansion joints are generally categorized as either nonseismic or seismic. The nonseismic joint widths are designed to accommodate contraction and expansion due to the VC effects and wind sway. For nonseismic expansion joints, the separation gap widths generally range between 1 to 4 in. (25 to 100 mm). On the other hand, seismic joints are designed to avoid seismic pounding during an earthquake event. Until 1997, the use of seismic joints was limited to structures located in high seismic zones such as California and the West Coast. After the new U.S. Geological Survey seismic hazard maps became the design basis, however, seismic design provisions became applicable to many jurisdictions in the U.S. As a result, the number of structures requiring seismic expansion joints has increased dramatically. Generally, seismic expansion joints are wider than nonseismic expansion joints, with widths ranging from 4 to 24 in. (100 to 600 mm) and more. In terms of cost, seismic joints cause more loss of usable space and are more expensive to install than the nonseismic expansion joints. Therefore, attention should be given to determine the gap width to keep it as narrow as possible. In this regard, it is also common practice to increase the gap width from footing to roof level to accommodate larger drifts at upper floors than on the lower floors, as shown in Fig. 4(a).

The most significant concern in determining a seismic joint width is seismic pounding. In cases where adjoining structures do not have sufficient separation, they may pound against each other during a seismic event. There have been several incidents of pounding during previous earthquakes, including the 1994 Northridge Earthquake that caused damage to several buildings.[2,3] Therefore, building codes require that two structural units across an expansion joint should be "separated structurally by a distance sufficient to avoid damaging contact under total deflection."[4,5] Determining an optimal seismic separation distance between adjoining structures, however, is not a straight-forward task.[6-8] Further, concrete is a nonlinear material that undergoes C-S-T-related deformations and affects the gap width over its service life.[9] Therefore, an expansion joint opening cast with a certain initial gap width does not retain the original width, but its width changes with time, temperature, and post-tensioning stress level in the adjoining structures. In proverbial terms, the separation gap tends to "grow" with time. The growth tendency needs to be considered in the expansion joint design.

This study undertakes a normative inquiry into the design of seismic joint gap width, incorporating VC effects to optimize the separation gaps to avoid seismic pounding in concrete parking structures. The design method presented herein is not limited to parking structures and can be used to compute separation gaps between concrete buildings.

STRUCTURAL POUNDING

During a seismic event, structures sway and drift. Two adjacent structures may drift in-phase or out-of-phase as shown in Fig. 4(a) through (c). The in-phase drift means that both structures drift in the same direction simultaneously so that the separation gap remains the same. If the structures acting as independent units swing out-of-phase, the gap opens and closes during a seismic event. In cases where joint width is inadequate, the structures pound against each other and thus have a damaging contact between them. The damaging contact is defined as the contact when two adjoining structures touch and impact each other or swing so close to each other that components housed in the gap can no longer function.

The structural pounding phenomenon involves two adjoining structures that relatively move toward each other and impact each other. After the impact, the structures stay in contact for a short duration and then separate. During the contact, a redistribution of momentum takes place and a part of kinetic energy is dissipated as heat, yielding, or crushing of the contact area.[2] The likelihood of pounding depends on fundamental periods of the structures. The energy dissipation depends on several factors, such as damping ratios and restitution ratios of the adjoining structures. Structural pounding is a nonlinear phenomenon. Several analytical studies have been reported to evaluate the effects of structural pounding. The studies show that both participating structures are affected by pounding, but to a varying degree.[2,3,10,11] The findings are summarized in the following:

1. Pounding is caused by out-of-phase vibration of adjacent structures, which depends on fundamental periods of the adjoining structures. The adjacent structures' fundamental period ratio T_B/T_A is a significant parameter. The subscripts A and B refer to two adjacent structures A and B, respectively. The period T_A is the larger of the two periods so that the ratio varies between zero and unity.

2. Two adjoining structures with T_B/T_A 0.3 show considerable out-of-phase drift and are very likely to pound against each other if their separation gap is inadequate. The structures with T_B/T_A between 0.3 and 0.7 exhibit some out-of-phase drift and are likely to pound against each other. The structures with T_B/T_A between 0.7 and unity drift essentially in-phase and are least likely to pound against each other.

3. Pounding can amplify the global response of participating structures in that it produces an acceleration response and shear force at various story levels that are greater than those with no pounding.

4. At the pounding story level, pounding occurs in the form of short duration spikes and influences the distribution of story peak responses throughout the building height. The response at the pounding level is more likely to cause malfunction of secondary systems at the pounding level than on other floors.

5. In the event of pounding between a stiffer building and a flexible building, pounding affects the flexible building more significantly, leading to permanent deformations of the flexible building.

6. The impact effect decreases as the separation gap between the adjoining structures increases and the ability of the structures to vibrate without pounding increases.

While pounding damage has been observed during several past earthquakes and analytical studies show the perils of pounding, Searer and Friedman[12] theorize that pounding of adjacent structures with coplanar floors may be advantageous because "the two adjoining structures will have a more difficult time resonating with the earthquake," and that "pounding will tend to damp out vibrations and reduce the responses of the two structures." On the other hand, they concede that "significant localized damage between the structures" is likely to occur due to pounding. The concept is meritorious but requires provision of a certain sacrificial slab width along the length of the expansion joint, which would get damaged and crumble during the design earthquake while the main structures would not sustain the pounding damage. Due to uncertainties and associated potential damage, the seismic pounding is considered an undesirable phenomenon and the best option is to provide an adequate separation gap at all elevated floor levels to avoid the damaging contact between the adjoining structures. The gap width needs to be kept to a minimum, however, to avoid loss of valuable real estate. Further, the larger the gap width, the more expensive it is to procure and maintain the hardware. Safety and repairability after an earthquake event pose additional concerns. The expansion joints are considered as the most delicate part in a structure; and it is foreseeable that the expansion joint cover plates would dislodge during an earthquake event leaving the gap wide open, creating a hazard for motorists, pedestrians, ambulances, and firefighters and causing disruption in the evacuation route. Further, in retrofitting of structures where widening of an existing expansion joint gap creates undue hardship for the owner, it is imperative to explore refined methods to optimize the expansion joint width design. The following section presents a state-of-the art overview of design methods available to determine the width of a seismic joint.

DESIGN FACTORS

The expansion joint sizing design involves determination of an optimal gap width to accommodate sufficient seismic separation. The optimal joint width is the shortest clear separation distance between two adjoining structures installed at the time of casting concrete so that the structures remain separate and independent without having a damaging contact between them during a seismic event. The design factors are:

1. Calculated seismic drift;
2. Type of expansion joint assembly selected; and
3. VC movements.

The first factor concerns the relative drift between two adjacent buildings during an earthquake and is the most significant factor in determining the separation width. The second factor concerns the hardware assembly provided. The third factor is used to further optimize the gap width. The factors are discussed in the following.

Seismic drift

A basic requirement for the separation width design is that the gap should be wide enough to avoid structural pounding, but not too wide to render the joint an unnecessary economic burden. IBC-2009 stipulates that the maximum inelastic response displacement should be calculated using the reduced lateral forces, and the displacements thus calculated should be scaled up by the deflection amplification factor C_d.[13] The C_d factor significantly affects the expansion joint gap width. The C_d factor varies with two factors: response modification coefficient R and the elastic response drift. The determination of both factors required considerable judgment based on knowledge of actual earthquake performance as well as research studies of various building systems.[14] Invariably, the C_d factor varies with the displacement ductility of the structural system being used. For example, the C_d factor for ordinary reinforced moment frame is 2.5, while it is 5.5 for special reinforced concrete moment frame.

Generally, upper bound of the separation distance u_{rel}, is obtained by assuming a full out-of-phase sway of the adjacent structures (Fig. 4(a) and (b)). It is calculated by adding the absolute seismic movement of each structure (ABS) as follows

$$u_{rel} = |u_A| + |u_B| \tag{1}$$

The estimated design earthquake displacements u_A and u_B can be obtained from the response spectrum. The subscripts A and B refer to structural units A and B, respectively. Alternatively, u_A and u_B can be determined by multiplying the displacements determined using code-prescribed static forces with the respective C_d factor. Kasai and Tran[7*] compared the ABS method with time-history analysis and determined that the ABS method is "excessively conservative," especially when buildings exhibited large inelastic deformations.For example, the separation requirement using the ABS method was up to 5.58 times more than one obtained using the time-history method. The reason is that the ABS method does not consider several key parameters affecting relative displacement between adjoining structures. Thus, the use of ABS results in the widest separation gap, which wastes underlying real estate.[7] Where a property line runs between the adjoining structures or where greater certainty against pounding is required, however, ABS is an appropriate method to compute the separation distance.

The required separation distance may also be calculated using the square root of sum of squares (SRSS) of the estimated earthquake displacements of the adjoining structural segments. Using the SRSS method, the seismic separation distance u_{rel} is computed as follows

$$u_{rel} = \sqrt{u_A^2 + u_B^2} \tag{2}$$

The SRSS method was prescribed in the International Building Code (UBC-1997)[15], but later versions of the code did not prescribe it until recently when IBC-2009 recodified the SRSS method as one of the methods to compute the building separation distance.

Studies[7] have shown that the ABS and SRSS methods estimate u_{rel} poorly, especially for a strong earthquake or when the natural periods of adjacent structures are close to each other, that is, when the ratio T_B/T_A is near unity and in-phase swings dominate. In such cases, the ABS and SRSS methods require conservative separation gaps, resulting in unnecessary construction and maintenance costs.

A more accurate method than the ABS and SRSS methods is called the spectral difference (SPD) method.[6-8] Kasai et al.[6] presented the SPD method using an elastic response spectrum to estimate the minimum relative displacement with the assumption that the adjacent structures are subject to identical ground motion simultaneously. Later, Kasai et al.[6] refined the method by using the elastic spectrum to approximate inelastic displacement of each structure at the expansion joint. Using the SPD rule, the peak relative displacement u_{rel} is given by

$$u_{rel} = \sqrt{u_A^2 + u_B^2 - 2\rho_{AB} u_A u_B} \tag{3}$$

where u_A and u_B are the peak drift values as defined previously and ρ_{AB} is the cross-correlation coefficient and reflects the vibration phase between the two buildings or structural units separated by an expansion joint. For two

*The term "conservative" is being used here in a relative sense with the time-history analysis assumed as the "exact" benchmark. It should be realized that the earthquake intensity at a site may exceed the anticipated design earthquake.

systems having equal damping ratios ρ_{AB} is given by

$$\rho_{AB} = \frac{8\xi^2(1 + T_B/T_A)(T_B/T_A)^{3/2}}{[1 - (T_B/T_A)^2]^2 + 4\xi^2(1 + T_B/T_A)^2(T_B/T_A)} \quad (4)$$

where T_A and T_B are the fundamental periods of the adjacent structural units A and B, respectively, and ξ is the average effective damping ratio of the structural units A and B.† As plotted in Fig. 5, ρ_{AB} varies with the period ratio T_A/T_B and damping of the buildings. When the structural units have identical periods and the ratio T_B/T_A is unity, the correlation coefficient ρ_{AB} becomes unity, regardless of the damping ratio. Larger ρ_{AB} means more in-phase motion and lower ρ_{AB} means more out-of-phase motion. For structural units that have equal periods and undergo equal displacements, ρ_{AB} becomes unity and the relative displacement u_{rel} reduces to zero, as shown in Fig. 4(c). It means that little separation is required to preclude pounding. Recognizing the effects of inelastic response and hysteretic behavior of adjoining structures on u_{rel}, Kasai and Tran[7] used displacement ductility to modify elastic fundamental periods and damping ratios in Eq. (4) using the following expressions

Bilinear hysteresis: $$T^* = T[1 + 0.09(\mu - 1)] \quad (5a)$$

$$\xi^* = \xi + 0.084(\mu - 1)^{1.3} \quad (5b)$$

Degrading hysteresis: $$T^* = T[1 + 0.18(\mu - 1) \quad (6a)$$

$$\xi^* = \xi + 0.084\,(\mu - 1)^{0.9} \quad (6b)$$

where T, ξ, and μ are the initial elastic vibration period, initial viscous damping ratio, and peak displacement ductility demand, and T^* and ξ^* are the modified period and damping ratio due to inelastic deformation, respectively.

To ascertain greater confidence in design of the separation gap, Kasai and Tran[7] suggested that the SPD-based u_{rel} value may be multiplied by a factor γ, which varies from 1.2 for a moderate (0.2 g) to 1.45 for a catastrophic (0.8 g) level earthquake. A linear interpolation may be used for other earthquake intensity levels. Statistically, the gap width thus determined would be wider than the "exact" time-history analysis with more than 85% confidence.[7] An example at the end of this paper illustrates the use of ABS, SRSS, and SPD methods in expansion joint design.

It should be pointed out, however, that the methods generally used to determine T and ξ are based on a variety of assumptions that cannot be readily verified

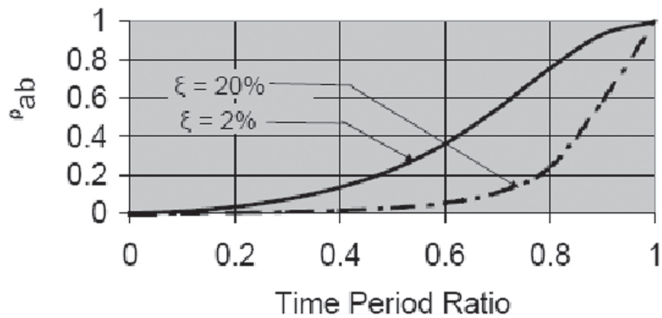

Fig. 5—Correlation factor using SPD method.

and, thus, their values are imprecise. Although the methods lend an appearance of accuracy to values thus generated, they may overstate the accuracy of the process. The accuracy in determining an optimal gap may be further compromised for structures of different material types and uses where the same assumptions are not applied to both structures. For example, if an engineer's methodology overstates the stiffness of a steel office building and understates the stiffness of an adjacent concrete parking structure, an unconservative result could occur.

Expansion joint assemblies

The expansion joint assemblies, also known as expansion joint systems, are specified to bridge the gap in the joint opening. An assembly may comprise several components, such as cover plate, nosing, gland, gutter, drain tube, and fire insulation. Some components of the system are located flush with, or at the top surface of, the slab and provide the bearing surface to vehicles and pedestrians crossing it, whereas other components are housed in the gap, as shown in Fig. 3. The components housed between the vertical walls occupy a part of the gap width and reduce the joint's working range. Therefore, the minimum width needed for proper functioning of expansion joint assemblies w_{ej} should be considered in design of joint width.‡ The magnitude of width w_{ej} depends on the type of expansion joint system specified and installed. Some systems require little width to function so that they keep functioning until the expansion joint side walls come in contact. Other systems, however, may need an inch or more in width to function in a fully closed state.[16] Therefore, the width w_{ej} should be added to u_{rel} when calculating the gap width

†A more general form of Eq. (4), which can handle unequal damping ratios in the structural units A and B, is given in References 6 through 8.
‡The width w_{ej} is defined as the minimum joint width or the narrowest linear gap a joint system tolerates and in which it performs its designed function without damaging its functional capabilities. For example, for LymTal Iso-Flex Xtreme Seismic Joint (with recessed plate), the w_{ej} is 1.5 in. (38 mm) as shown in Reference 16.

needed to avoid damaging contact between the adjacent structures.

VC effects

VC movement starts on the day concrete is placed. Its effects on the separation gap width can be divided into two parts: (a) during the construction phase before the joint assembly has been installed; and (b) after the assembly is installed and thereafter during the service life of the structure. The VC movement Δ_{vc} can be calculated using the following equation

$$\Delta_{vc} = M(\Delta_{cr} + \Delta_{sh} + \Delta_{a} + \Delta_{t}) \tag{7}$$

where the terms Δ_{cr}, Δ_{sh}, and Δ_{a} represent structural shortening due to creep, shrinkage, and post-tensioning axial compression, respectively. The term Δ_{t} represents thermal movement. ACI-209R provides information on predicting shrinkage and creep of concrete.[9] The movement factor M has been determined to be 0.6 for pretopped precast parking structures and 0.8 for post-tensioned structures.[1] It is well established that a significant part of creep and shrinkage takes place during the days after concrete is placed. The drying shrinkage of concrete during the first year of concrete life is estimated to be 511 μstrain.[17] Considering e_{th} = 7.5 μstrain/°F (4.2 μstrain/°C), it will take a temperature rise of 68 °F (38 °C) to compensate for the shrinkage. Similarly, significant concrete creep occurs in the first year, causing additional shortening.[17] Thus, the combined effect of creep and shrinkage is larger than the thermal expansion, thermal expansion is consumed in offsetting a part of creep and shrinkage effects, and the structure shortens due to creep and shrinkage, and the separation gap increases as it ages. The post-tensioning axial force further adds to the shortening and the net result is that the gap widens. Widening the gap between adjoining structures due to VC movement can help avoid seismic pounding. Due to the unpredictability of an earthquake event, however, it cannot be readily relied on or used in the gap design to avoid seismic pounding. On the other hand, VC effects need to be considered in expansion joint design. Therefore, the VC movement is divided into two time segments: (a) before the expansion joint hardware is installed (0 to t days); and (b) after the hardware is installed (t to ∞ days). The effects are described in the following.

VC effects before installation of joint assembly

The expansion joint assemblies are installed when a parking structure is substantially complete and is nearing occupancy. The construction process typically takes a year or longer. The lower floors are built first; therefore, the expansion joints at the lower floors stay open longer than the upper floors before the expansion joint assemblies are installed. During this time, the structure undergoes VC movements and the gap installed at the time of concrete placement widens by an increment, say, Δ_{vc0}. Therefore, the initial width S_0 to be cast can be determined using the following equation

$$S_0 = u_{rel} + w_{ej} - \Delta_{vc0} \tag{8}$$

The VC increment Δ_{vc0} represents the increase in expansion joint gap width due to three shortening factors: creep, shrinkage, and post-tensioning shortening effects before the joint assembly is installed. The changes due to thermal movements are generally neglected when calculating Δ_{vc0} because it is hard to predict the temperature on the day the expansion joint assembly will be installed. The temperature effect, however, should be considered by adjusting the gap on the day the assembly is being installed.

VC effects after installation of joint assembly

After an expansion joint assembly is installed in a newly built structure, the structure continues to undergo VC movements. The movements take place at an asymptotically slower rate and the gap widens by an increment, say, Δ_{vc1}. The maximum expansion joint width in a fully open position during a seismic event S_{max} is determined as follows

$$S_{max} = S_0 + u_{rel} + \Delta_{vc1} \tag{9}$$

The term Δ_{vc1} represents an increase in the joint gap width due to diaphragm shortening due to three shortening factors: creep, shrinkage, and temperature. It covers the time segment after the joint assembly is installed. The VC movement widens the separation gap; however, due to unpredictability of a seismic event, it cannot be used in avoiding seismic pounding. On the other hand, Δ_{vc1} affects the swinging range of the expansion joint and, therefore, needs to be considered in the hardware or assembly design. For example, the assemblies using metal plates to carry traffic loads (Fig. 3) need to have adequate plate bearing widths on both sides of the expansion joint to avoid the plates falling into the gap during a seismic event. The wider a gap gets, the lesser is the plate bearing width available. Therefore, the VC movement Δ_{vc1} needs to be considered to determine S_{max}.

Table 1—Seismic joint design in in. (mm)

Level	δ_A	δ_B	u_A	u_B	u_{rel} ABS	u_{rel} SRSS	u_{rel} SPD	Age at installation t (days)	Δ_{vc0}	Δ_{vc1}	w_{ej}	S_0	S_{max}
6	0.81 (21)	0.52 (13)	3.62 (92)	2.32 (59)	5.94 (151)	4.3 (109)	3.43 (87)	30	0.56 (14)	1.25 (32)	1.5 (38)	4.37 (111)	9.06 (230)
5	0.79 (20)	0.50 (13)	3.54 (90)	2.23 (57)	5.77 (147)	4.18 (106)	3.34 (85)	60	0.76 (19)	1.05 (27)	1.5 (38)	4.09 (104)	8.48 (215)
4	0.74 (19)	0.46 (12)	3.35 (85)	2.08 (53)	5.43 (138)	3.94 (100)	3.16 (80)	90	0.87 (22)	0.93 (24)	1.5 (38)	3.78 (96)	7.87 (200)
3	0.71 (18)	0.42 (11)	3.2 (81)	1.89 (48)	5.09 (129)	3.71 (94)	2.99 (76)	120	0.95 (24)	0.85 (22)	1.5 (38)	3.54 (90)	7.38 (187)
2	0.64 (16)	0.36 (9)	2.89 (73)	1.61 (41)	4.49 (114)	3.30 (84)	2.68 (68)	150	1.01 (26)	0.79 (20)	1.5 (38)	3.17 (81)	6.64 (169)
Ground	—	—	—	—	—	—	—	—	—	—	—	—	—

Note: SPD method used to determine S_0 and S_{max}.

In summary, expansion joint width design requires the determination of two gap widths, namely, S_0 and S_{max}. The steps needed to design the widths are illustrated in the following.

DESIGN EXAMPLE

The objective of this example is to demonstrate the design of an expansion joint between two portions of a post-tensioned parking structure, such as that shown in Fig. 2, located in the western region of the U.S. The assigned seismic category (SDC) is "C." The design temperature variation is 51 °F (10.5 °C). The relative humidity is 55%. The parking structure has five elevated levels, each with a 5.5 in. (140 mm) post-tensioned slab with 200 psi (1.38 MPa) effective prestress. It has been divided into two portions, A and B, having a length of 170 and 200 ft (53 and 62 m), respectively. The intermediate moment frames were used to resist the seismic load (C_d = 4.5). The fundamental periods T_A and T_B are 1.0 and 0.62 seconds, respectively, and the ratio T_B/T_A is 0.62. The drifts δ_A and δ_B were computed under IBC static forces using the ETABS computer program. The design earthquake displacements u_A and u_B at elevated levels were computed by multiplying δ_A and δ_B with factor C_d and are shown in Table 1. For intermediate moment frames, use $\mu = 3.5$ ($= C_d - 1$) and elastic damping ratio ξ = 2% for both portions. Using Eq. (6a) and (b), the inelastic periods ratio remains unchanged and the inelastic damping ratio ξ^* = 21% for degrading hysteresis. Using Eq. (4), ρ_{AB} = 0.4. The relative earthquake drift u_{rel} using the ABS, SRSS, and SPD methods was computed. Table 1 shows that the SPD method yields the smallest relative displacement.

For the computed gap width range over the building height, using a steel plate expansion joint system, as shown in Fig. 3, appears appropriate. There are many products available on the market to bridge the separation gap. Here, the seismic joints in Reference 16 are being considered for final selection. Their properties and swing ranges are available on the Internet. The width w_{ej} for the selected expansion joint type is 1.5 in. (38 mm).[16]

The determination of VC effects is a two-step process, requiring determination of Δ_{vc0} and Δ_{vc1} factors. The Δ_{vc0} factor is used to calculate diaphragm shortening due to creep, shrinkage, and post-tensioning before the expansion joint is installed. The Δ_{vc1} factor is used to calculate diaphragm shortening due to creep, shrinkage, and temperature after the expansion joint is installed. For this example, the age of the concrete diaphragm at which the expansion joint is installed varies from one floor to another, as shown in Table 1. For the top floor, the age of the diaphragm is assumed to be 30 days when the expansion joint assembly would be installed. The corresponding factors are computed as follows

$$\Delta_{vc0}\ (t = 0 \text{ to } 30) = M(\Delta_{cr} + \Delta_{sh} + \Delta_a) = (0.8)(185 \times 12)(40 + 205 + 69) \times 10^{-6} = 0.56 \text{ in. } (14.2 \text{ mm})$$

$$\Delta_{vc1}\ (t = 30 \text{ to } \infty) = M(\Delta_{cr} + \Delta_{sh} + \Delta_t) = (0.8)(185 \times 12)(83 + 390 + 230) \times 10^{-6} = 1.25 \text{ in. } (31.7 \text{ mm})$$

The factor M is the movement factor for post-tensioned structures. The tributary length for the VC movements was determined to be one-half of the parking facility's length. The VC strains are computed using ACI 209R-92[9] equations. The modulus of thermal expansion of 7.5 microns per °F was used in calculating the temperature movement. As shown in Table 1, the maximum required opening S_{max} is 8.56 in. (217 mm).

For this, a suitable assembly is a seismic expansion joint system[15] with a maximum opening of 9 in. (228.6 mm). A width of 3.87 in. (98.3 mm) is required for the initial gap S_0. The same procedure is repeated to design expansion joints at all other levels.

SUMMARY AND RECOMMENDATIONS

Three methods to compute the expansion joint widths to avoid damaging contact through seismic pounding have been presented. The ABS method results in the widest joint and the SPD method results in the narrowest joints. The situations in which the methods can be used are explained in the paper. The final choice of design method depends on the engineer's intended performance goals. In an essential facility, such as a hospital, the engineer may use the ABS method to provide a high degree of certainty. In a retrofit project or where damage does not appear likely to be a potential initiator of collapse, the SPD method may be more appropriate. The SPD method optimizes the joint opening in keeping the joints narrow and yet avoids seismic pounding. Additional research is needed to determine the effect of pounding in concrete structures, particularly those having coplanar floors and little nonstructural elements. The restitution value and impact duration should be determined for a head-on collision between the adjacent diaphragms during a seismic event. The effect of sacrificial slab damage should also be investigated. Such research will help reduce the expansion joint gap width and benefit all involved in the design, construction, and maintenance of parking structures and other buildings.

NOTATION

T_A, T_B = fundamental period of structural units A and B, respectively

u_A, u_B = estimated design earthquake displacements of structural units A and B, respectively

u_{rel} = relative displacement of two adjacent structural units during a seismic event

Δ_T = design temperature

Δ_{vc0}, Δ_{vc1} = expansion joint width changes due to volume change effects

δ_A, δ_B = lateral drift under code-specified forces for units A and B, respectively

γ = factor used in SPD method to reflect the earthquake

μ = displacement ductility ratio

ρ_{AB} = cross-correlation coefficient for adjoining structural units A and B

ξ = average damping of adjoining structural units

REFERENCES

1. Iqbal, M., "Thermal Movements in Parking Structures," *ACI Structural Journal*, V. 104, No. 5, Sept.-Oct. 2007, pp. 542-548.

2. Malhotra, P. K., "Dynamics of Seismic Pounding at Expansion Joints of Concrete Bridges," *Journal of Engineering Mechanics*, ASCE, V. 124, No. 7, July 1998, pp. 794-802.

3. Abdel Raheem, A. E., "Seismic Pounding between Adjacent Building Structures," *Electronic Journal of Structural Engineering*, eJSE, V. 6, 2006, pp. 66-73.

4. "International Building Code," International Code Council, Falls Church, VA, 2006.

5. ASCE/SEI 7-05 Committee, "Minimum Design Loads for Buildings and Other Structures," American Society of Civil Engineers, Reston, VA, 2005, 388 pp.

6. Kasai, K.; Jagiasi, A. R.; and Jeng, V., "Inelastic Vibration Phase Theory for Seismic Pounding Mitigation," *Journal of Structural Engineering*, Oct. 1996, pp. 1136-1146.

7. Kasai, K., and Tran, B. T., "A Simplified Method to Predict Peak Value and Trend of Seismic Relative Motion between Adjacent Buildings," *Journal of Structural and Construction Engineering, Architectural Institute of Japan*, No. 582, 2004, pp. 47-55.

8. Garcia, D. L., "Separation between Adjacent Nonlinear Structures for Prevention of Seismic Pounding," *Paper* No. 478, Proceedings of the 13th World Conference on Earthquake Engineering, Vancouver, BC, Canada, Aug. 1-6, 2004.

9. ACI Committee 209, "Prediction of Creep, Shrinkage and Temperature Effects in Concrete Structures," 209R-92 (Reapproved 1997), American Concrete Institute, Farmington Hills, MI, 47 pp.

10. Muthukumar, S., and DesRoches, R., "A Hertz Model with Non-Linear Damping for Pounding Simulation," *Earthquake Engineering and Structural Dynamics*, John Wiley & Sons, 2006, V. 35, pp. 811-828. (www.interscience.wiley.com)

11. Jankowski, R., "Non-Linear Viscoelastic Model of Structural Pounding," *Proceedings*, 23rd World Conference on Earthquake Engineering, Vancouver, BC, Canada, Aug. 2004.

12. Searer, G. R., and Freeman, S. A., "Design Drift Requirements for Long-Period Structures," *Paper* No. 3292, Proceedings of the 13th World Conference on Earthquake Engineering, Vancouver, BC, Canada,

Aug. 2004.

13. "International Building Code," International Code Council, Washington, DC, 2009.

14. Building Seismic Safety Council, "NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures," Part 2: Commentary (FEMA 303), Feb. 1998.

15. "Uniform Building Code (UBC-1997)," V. 2, Structural Engineering Design Provisions, International Code Council, 1997.

16. http://www.lymtal.com/drawing/1084_draw.pdf (last accessed July 7, 2009).

17. Prestressed Concrete Institute, *PCI Design Handbook*, sixth edition, PCI, Chicago, IL, 2005, 750 pp.

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