ROTATIONAL MEASUREMENTS IN STRUCTURES – WHY AND HOW? - ENGINEERS' PERSPECTIVE AND EXPERIENCE

by

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SUMMARY

Complete time-space characterization of a point on a deformable body is achieved by measuring or theoretically including three translational (*x, y* and *z*) and three rotational (θ_x, θ_y and θ_z) degrees of freedom in the Cartesian coordinate system. Whether it is the Earth's surface or a structural system, this generic concept remains unchanged. However, traditionally, ground motion measurements have been limited to three translational components. On the other hand, during design and analysis processes and in laboratory testing of structural members, components or systems, engineers have been measuring, assessing and computing rotations in addition to translational deformations. The significant actions caused by rotations of sections, members, joints, and a structure as a whole describe in fact the behaviour and, in turn, performance state of a structure. For bending, torsional, twisting, rocking and other important behavior of a structure, rotations are the main variables. Therefore, under seismic and gravity loads, measurements of rotation in instrumented structures have paramount importance to better understand and assess the deformational behavior for performance evaluation of the structural systems. In this paper, examples of quantification of rotational behavior are provided from data retrieved from instrumented structures during strong shaking events. It is shown that rotations are computed from actual measurements of displacements computed from accelerations recorded with uniaxial translational accelerometers. In the vast majority of instrumented structures, to date, rotational sensors have not been utilized. For example, as of writing of this paper, the authors are not aware of any building that has utilized rotational sensors in its instrument arrays. Despite this deficiency, it is shown, in all presented cases, that rotations can be computed from measurements of uni-directional translational sensors.

INTRODUCTION

In structural engineering, during iterative design and analyses processes, several mathematical modeling assumptions are made considerate of actual structural behavior under gravity and lateral load or combinations thereof. In theory, while the deformation of every point in a structure is represented by six-degrees of freedom (DOF), when modeling, engineers make simplifications based on knowledge of behavior of structural elements, members, joints, components and the structural system as a whole. For example, in finite element analyses, axial deformation of beam members, represented as beam elements, are neglected but, for column members, axial deformation is not always neglected - depending on the number of stories or how slender the columns are. For taller buildings, axial deformations become important because of P-delta effect (e.g., destabilizing effect of axial load associated with lateral deformations). In most cases, for low-rise framed buildings, axial deformations of beam or columns are neglected. On the other hand, floor slabs, if represented by beam elements, axial deformations are not included but, if represented in a three-dimensional (3-D) model as plate or shell elements, in-plane deformations are automatically included because of the nature of finite element formulation for those types of elements. In addition, rotations at joints, particularly for frame-type structural systems are almost always included. Neglecting joint rotations may cause gross errors in computation of displacements and stresses. All civil/structural engineers learn these from structural theory and analyses as well as engineering practice or research.

While the above discussion provides a very gross simplification of issues related to mathematical modeling, similar considerations must be made when monitoring seismic responses of structural systems. In other words, measurements must accurately reflect and include essential deformations (in general displacements and rotations) in order to fulfill one of the key objectives for monitoring that an instrumented structure should provide enough information to reconstruct the response of the structure in sufficient detail to compare with the response predicted by mathematical models and those observed in laboratories, the goal being to improve the models (Çelebi, 2004). Therefore, before going further, it is important to review general objectives and practices for monitoring/measuring deformations in structures. The scope of this paper is limited to monitoring for seismic responses only and does not include monitoring for other purposes (*e.g.* environmental).

MONITORING STRUCTURES – STATE OF PRACTICE

Instrumentation and Data Utilization

Ultimately, the types and extent of instrumentation must be tailored to how the acquired data will be utilized, even though there may be more than one objective for instrumenting a structure. Table 1 summarizes some data utilization objectives with sample references.

Instrumentation – General Schemes and Code Requirements

The most widely used code in the United States is the Uniform Building Code (UBC-1997 and prior editions), which recommends that for seismic zones 3 and 4, a minimum of three accelerographs be placed in every building over six stories having an aggregate floor area of 60,000 square feet or more, and in every building over ten stories regardless of the floor area. The purpose of this requirement by the UBC is to monitor gross response rather than to analyze the complete response modes and characteristics.

Table 1. Sample List of Data Utilization Objectives & Sample References

GENERIC UTILIZATION
Verification of mathematical models (usually routinely performed) (*e.g.* Boroschek et al, 1990)
Comparison of design criteria vs. actual response (usually routinely performed)
Verification of new guidelines and code provisions (*e.g.* Hamburger, 1997; Goel and Chopra, 2004 Kunnath et al., 2006; Kalkan and Kunnath, 2006a, 2007)
Identification of structural characteristics (Period, Damping, Mode Shapes)
Verification of maximum drift ratio (*e.g.* Astaneh, 1991; Çelebi, 1993)
Torsional response/Accidental torsional response (*e.g*. Chopra, 1991; De La Llera, 1995)
Identification of repair & retrofit needs & techniques (Crosby, 1994)
SPECIFIC UTILIZATION
Identification of damage and/or inelastic behavior (*e.g*. Rojahn and Mork, 1981)
Soil-Structure interaction including rocking and radiation damping (Çelebi, 1996, 1997)
Response of Unsymmetric Structures to Directivity of Ground Motions (*e.g.* Porter, 1996) Response of structures to near-fault pulses (e.g. Kalkan and Kunnath 2006b)
Responses of structures with emerging technologies (base-isolation, visco-elastic dampers, and combination (Kelly and Aiken, 1991; Kelly, 1993; Çelebi, 1995)
Structure specific behavior (*e.g*. diaphragm effects, Boroschek and Mahin,1991; Çelebi, 1994)
Development of new methods of instrumentation/hardware (e.g. GPS: Çelebi et. al., 1997, 1999, 2002; wireless: Straser, 1997)
Improvement of site-specific design response spectra and attenuation curves (Boore, et. Al. 1997; Campbell, 1997; Sadigh *et. Al*., 1997; Abrahamson and Silva, 1997; Graizer and Kalkan 2007)
Associated free-field records (if available) to assess site amplification, SSI and attenuation curves(Borcherdt, 1993, 1994, 2002a, 2002b; Crouse and MacGuire, 1996)
Verification of repair/retrofit methods (Crosby et al, 1994; Çelebi and Liu, 1996)
Identification of site frequency from building records (Çelebi, 2003)
RECENT TRENDS TO ADVANCE UTILIZATION
Studies of response of structures to long period motions (e.g. Hall et al, 1996)
Need for new techniques to acquire/disseminate data (Straser, 1997; Çelebi, 1998; Çelebi and Sanli, 2002; Çelebi et al., 2004) Impacts of rotational components and their coupling effects on structures (Kalkan and Graizer, 2007ab)
Verification of performance based design criteria (future essential instrumentation work)
Near fault factor (more free-field stations associated with structures needed)
Comparison of strong vs. weak response (Marshall, Phan and Çelebi, 1992, Çelebi, 1993)
Functionality Çelebi, 2004, Needs additional specific instrumentation planning)
Health monitoring and other special purpose verification (Heo et al, 1997)

UBC-code-type recommended instrumentation[1] is illustrated in Figure 1a. In general, code-type instrumentation is naturally being de-emphasized as a result of strong desire by the structural engineering community to gather more data from instrumented structures to perform more detailed structural response studies. Experiences from past earthquakes show that the minimum guidelines established by UBC for three tri-axial accelerographs in a building are not sufficient to perform meaningful model verifications. For example, three horizontal accelerometers are required to define the (two orthogonal translational and a

[1] Following 1971 San Fernando earthquake, in 1982, in Los Angeles, the code-type requirement was reduced to one tri-axial accelerometer at the roof (or top floor) of a building meeting the aforementioned size requirements (Darragh and others, 1994).

torsional) horizontal motions of a floor. Rojahn and Mathiesen (1977) concluded that the predominant response of a high-rise building can be described by the participation of the first four modes of each of the three types of motion (two translations and torsion); therefore, a minimum of 12 horizontal accelerometers would be necessary to record these modes. Instrumentation needed to provided acceptable documentation of the dominant response of a structure are addressed by Hart and Rojahn (1979) and Çelebi and others (1987). This type of instrumentation scheme is called the ideal extensive instrumentation scheme as illustrated in Figure 1b.

Specially designed instrumentation arrays are needed to understand and resolve specific response problems. For example, thorough measurements of in-plane diaphragm response require sensors in the center of the diaphragm (Figures 1c) as well as at boundary locations. Performance of base-isolated systems and effectiveness of the isolators are best captured by measuring tri-axial motions at the top and bottom of the isolators as well as the rest of the superstructure (Figure 1d). In the case of base-isolated buildings, the main objective usually is to assess and quantify the effectiveness of isolators. To the extent affordable, additional sensors can be deployed between the levels above the isolator and roof to capture the behavior of intermediate floors.

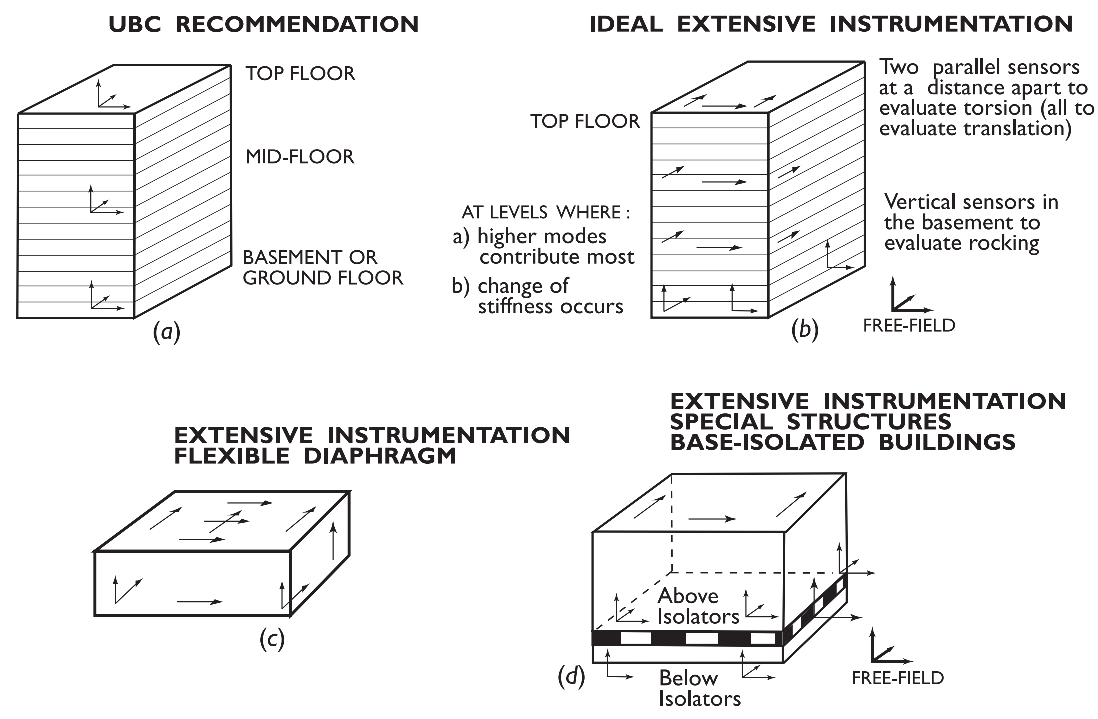

Figure 1. *Typical instrumentation schemes.*

In all these schematic cases and in actual applications to date, only very few rotational sensors have been deployed (e.g. Pezeshk and Withers, 2006). In the United States, neither CSMIP nor USGS, the largest network operators of structural monitoring systems, have deployed any rotational sensors. Rather, both strong-motion operating programs have relied on uniaxial sensors or combinations of uniaxial sensors to measure both translational and rotational motions from which the deformations (including displacements and rotations) have been computed.

Thus, it should be stressed that while rotational sensors are not deployed widely in engineering seismic monitoring applications, rotation as a form of deformation or displacement in a broader sense is not neglected. Quite to the contrary, measuring rotation is an important aspect of seismic monitoring. From a data utilization point of view, measuring rotation is in general useful for the following major reasons:

1. Torsional motions – both modal response and time-history response to man-made or natural excitation. Torsional motions can be caused by in-plan eccentricity or vertical irregularity associated with asymmetry, offsets, or abrupt changes in stiffness, strength, locations of mass center and/or centers of rigidity. It has been reported in many studies that torsional response is a critical mode triggering early failure in structures due to enhanced shear demand imposed on vertical load carrying system and beam-column connections (e.g., Kalkan and Kunnath 2006c). From analyses and design points of view, engineers seek to find out what additional stresses are imposed on the structure on top of the stresses caused by purely translational motions. Even on a geometrically symmetric structure, engineers impose "accidental eccentricity" to account for additional torsion due to possible lateral load application that may not be applied at the mass center.
2. Rocking, a form of soil-structure interaction (SSI), occurs when a structural response includes contributions from rigid body rotation of the system together with its foundation in addition to contributions from bending and shear deformations.
3. Rotation of basemat and basement walls is another SSI-induced phenomenon that occurs due to rigid response of structural system as opposed to soil deformation underneath the rigid basemat or foundation.
4. Rotation of a joint, structural component or element is the most important deformation that represents the response of a structure under gravity and/or lateral loads. For that reason, it is considered as one of the key engineering-demand parameters describing the performance state.

In the following sections, we examine rotation as the variable in the deformation and the consequence of behavior during the seismic response of a structure. Hereinafter, real-life response data from instrumented structures are provided as examples of measured rotations.

ROTATION AND BENDING

Measuring rotations is easier in the laboratory than in actual structures since rotations are measured in a limited number of locations (e.g. rotation or curvature measurement of a section of a beam or column member, gross rotation of a cantilever member with respect to a plastified "hinged" section, or "chord" rotation represented by relative displacements of a bending member between the two ends of a "beam element" divided by the length of that element). This latter form of deformation is particularly important for computing/assessing drift ratio of columns of a building structure under gravity and lateral load combinations and is extensively used to assess performance level of parts or whole structural system (Çelebi and Sanli, 2004). Thus, in the laboratory or in prototype structures, if relative displacement of a member is computed or measured correctly, it includes the effect of rotation of its ends, that is, the joints. The simple slope-deflection equation used for framed structures best describes the variables considered as:

$$M_{ij}=(2EI/L)[2\theta_i+\theta_j-3\Delta_{ij}/L] \tag{1}$$

where M_{ij} is the bending moment at end i of a beam-column member ij, θ_i is rotation at end i, θ_j is rotation at end j, Δ_{ij} is relative displacement between the ends of member ij, L is the length of the member, I is the moment of inertia of the member cross-section and E is the modulus of elasticity (Figure 2). Thus, solving Eq. 1 for Δ_{ij} yields,

$$\Delta_{ij}=L/3[2\theta_i+\theta_j-(M_{ij}L/2EI)] \tag{2}$$

Notably, Δ_{ij} is a function of M_{ij}, θ_i, θ_j and the member properties (L, E and I). Therefore, in the basic formulation or actual measurement of Δ_{ij},, rotations are already included. Hence, the goal should be to measure Δ_{ij}, as accurately as possible.

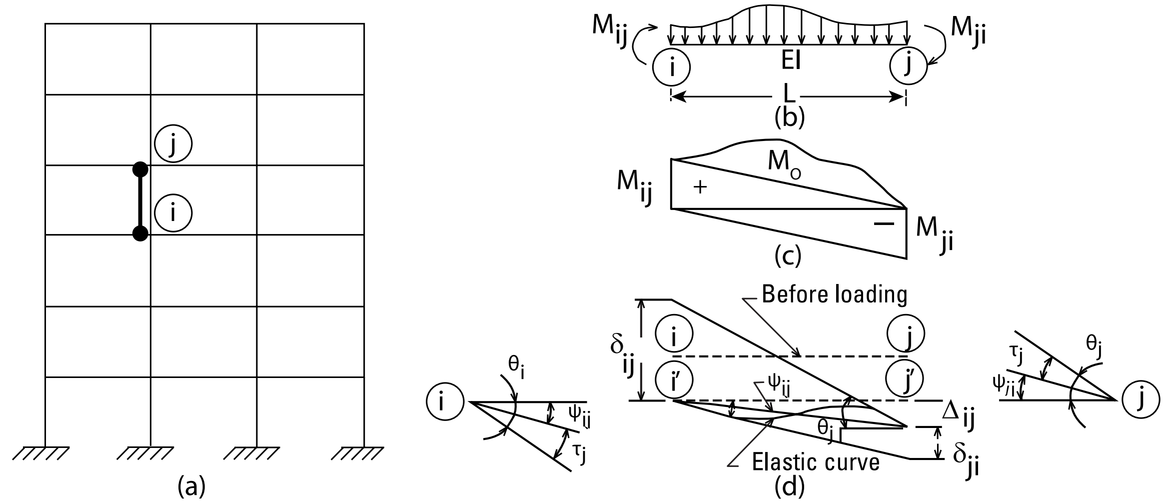

Figure 2. *Schematic showing structural mechanics of a column in bending and the variable – joint rotation.Shown are (a) a column member, ij, (b) member with end moments and loading, (c) member moment diagram, and (d) deformations considered.*

ROTATION AND TORSION

For buildings and most other structures, it is generally very difficult to vibrate in a purely torsional mode. Typically, torsional motions are coupled with translational motions. As such, measurements must be made such that the contribution of torsional motion to the overall total motion can be adequately quantified. As stated before, from analyses and design point of view, engineers seek to find out what additional stresses are imposed on the structure on top of the stresses caused by purely translational motions. Even on a geometrically symmetric structure, engineers impose "accidental eccentricity" to account for additional torsion due to possible lateral load application that may not be applied at the mass center.

In general, torsional vibration can be caused by three main factors:

1. In-plan eccentricity when center of mass center and centers of rigidity are not concurrent. This could be due to unsymmetrical plan geometry and/or unsymmetrical mass distribution in plan.
2. Vertical irregularity due to offsets and smooth or abrupt changes in stiffness, and/or mass. An example of this type is illustrated in Figure 3.
3. Application of earthquakes input motions at an eccentric distance from center of rigidity.

The in-plan eccentric and vertically irregular building shown in Figure 3 is in Los Angeles, CA. Figure 3 also shows acceleration records obtained during the 1987 Whittier, CA earthquake. The records manifest, on each of the three floors, the significant differences in parallel motions due to predominantly torsional behavior of the building (Çelebi, Safak and Youssef, 1991). The torsional response of the building is particularly interesting because it is the second torsional mode, as demonstrated in Figure 4. This quantification is extracted from data obtained from uniaxial translational accelerometers deployed parallel and apart from each other at each of the instrumented floors.

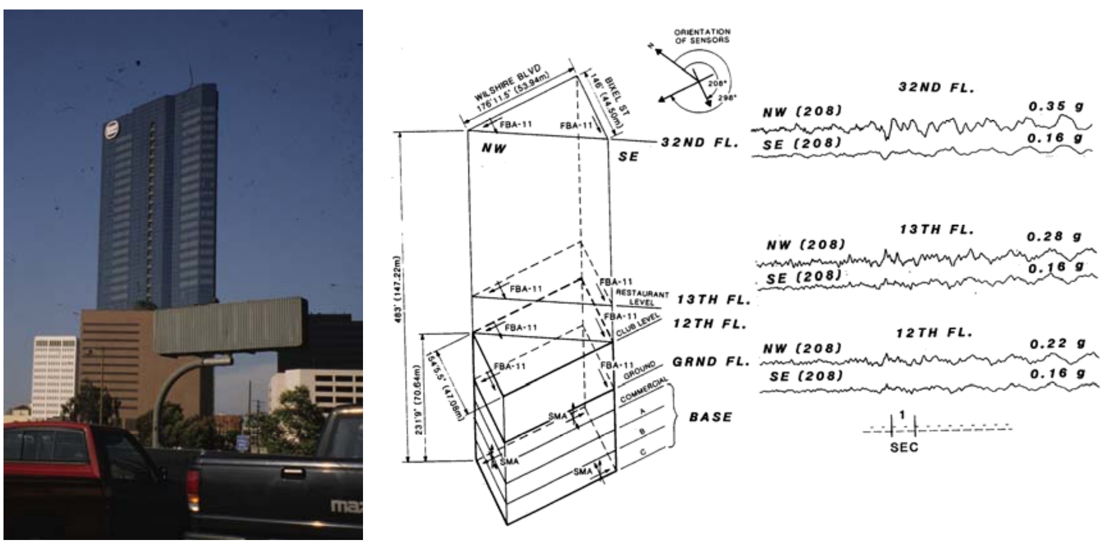

Figure 3. *Left: Vertically irregular building in Los Angeles. Twenty triangular stories sit on top of 12 rectangular stories (plus basement floors). Center: Instrumentation of the building. Right: Acceleration records obtained during 1987 Whittier, CA earthquake exhibit that parallel motions on three floors are different both in amplitude and content due to torsional response.*

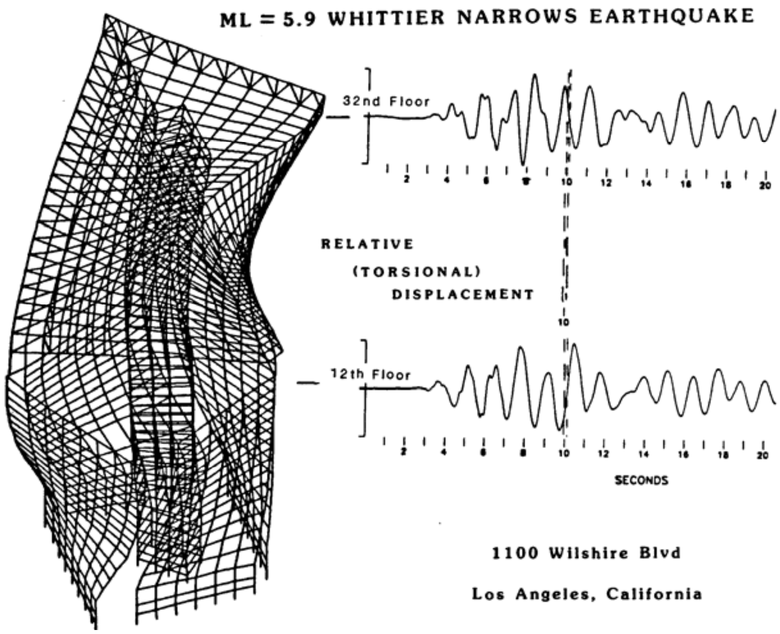

Figure 4. *Torsional displacements computed for two floors show that, as illustrated by phase difference, the second torsional mode is the dominant contributor to the overall response (Note: Deformed shape is exaggerated for illustration purposes)(Çelebi, Safak and Youssef, 1991).*

ROTATION AND TWISTING

Torsion of a structural system involves the response of the entire structure. However, some wings or ensemble of elements may twist (rotate) relative to a core or central area of a structure. This is particularly true for winged structures. A good example of this case is illustrated in Figure 5 which shows the Pacific Park Plaza (PPP) Building in Emeryville, CA. PPP is an equally-spaced, three-winged, cast in place, thirty-story, 312 ft. (95.1 m) tall, ductile reinforced concrete moment-resisting frame building. The three wings of the building are constructed monolithically and are equally spaced at angles of 120 degrees around a central core. Shear walls in the center core and wings extend to the second floor level only, but column lines are continuous from the foundation to the roof. The foundation is a 5-foot-thick concrete mat supported by 828 (14-inch-square) pre-stressed concrete friction piles, each 20-25 m in length, in a primarily soft-soil environment having an average shear-wave velocity between 250 and 300 m/s and a depth of approximately 150 ft (~50 m) to harder soil.

The seismic instrumentation of the PPP Building, also illustrated in the three-dimentional Figure 5, integrates arrays for the structure, surface, and downhole, and comprises a 30-channel accelerometer deployment uniquely designed to capture (a) the translational motions of the wings of the building relative to its core, (b) the vertical motions of the mat foundation slab at the ground floor level, and (c) free-field motions at the surface and at a downhole depth of 200 ft (61 m). The free-field site is often referred as the Emeryville (EMV) site.

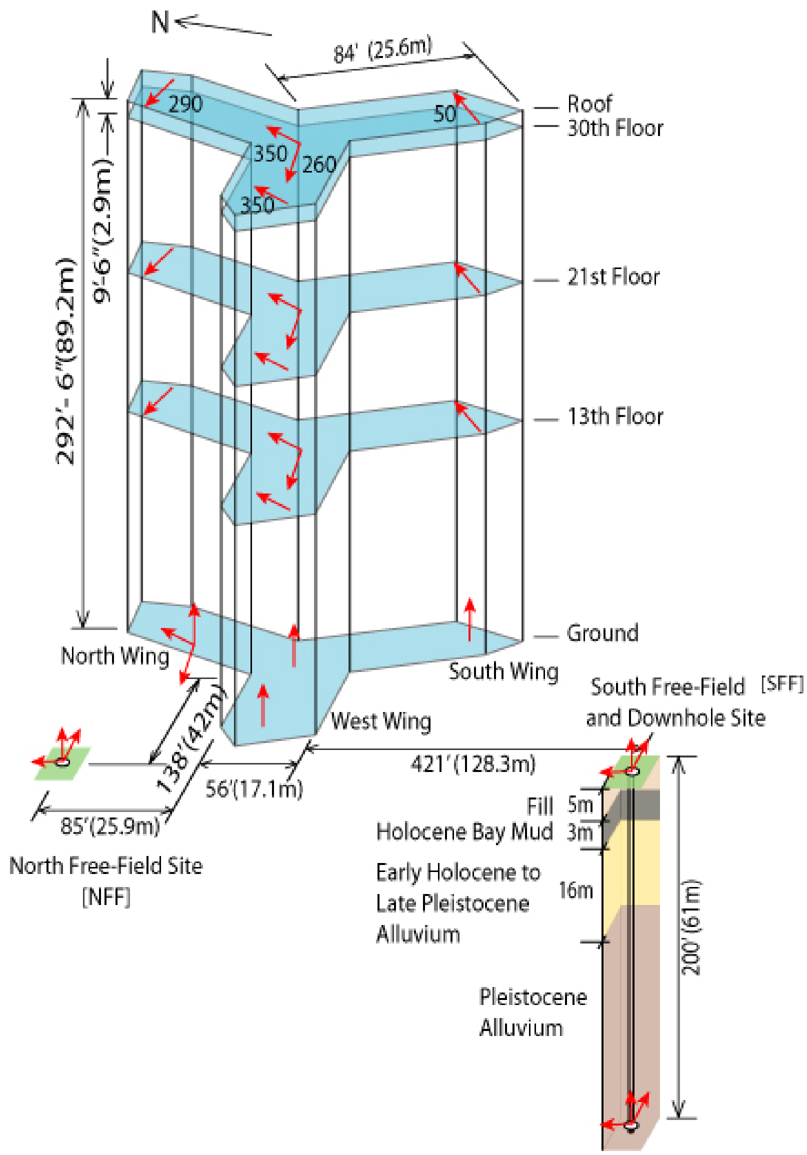

Figure 5. *A three-dimensional schematic of the Pacific Park Plaza Building (Emeryville, CA) showing integrated structure, surface and downhole sub-arrays (Note: The tri-axial downhole accelerograph was added after the 1989 Loma Prieta earthquake).*

Responses of the building and the surface free-field were recorded during the strong shaking caused by the 1989 Loma Prieta earthquake. The building was not damaged. Detailed analyses of complete set of data are reported elsewhere (Çelebi and Safak, 1992; Çelebi, 1998).

Figure 6 exhibits relative displacements of the wings compared to the ground floor – effectively representing twisting of the wings. The response of the building was also found to be sensitive to the dominant orientation of the maximum energy of LPE ground motions (Figure 6). The 290^{o} transverse orientation of the NW wing of the building was similar to the rupture direction of the earthquake (Çelebi, 1992). A significant effect of the orientation of the ground motion for an winged building such as the PPP is that it exhibited a disproportionate (as much as three times) response in one wing of the building compared to the others (Çelebi, 1992). Therefore, the propagation direction of different waves (in most cases, surface waves) arriving at a building can significantly affect building response. As a general conclusion, because the energy of the ground motions can be azimuthally variable, response of structures with wings or unsymmetrical structures can be significantly affected by it (Çelebi, 1992).

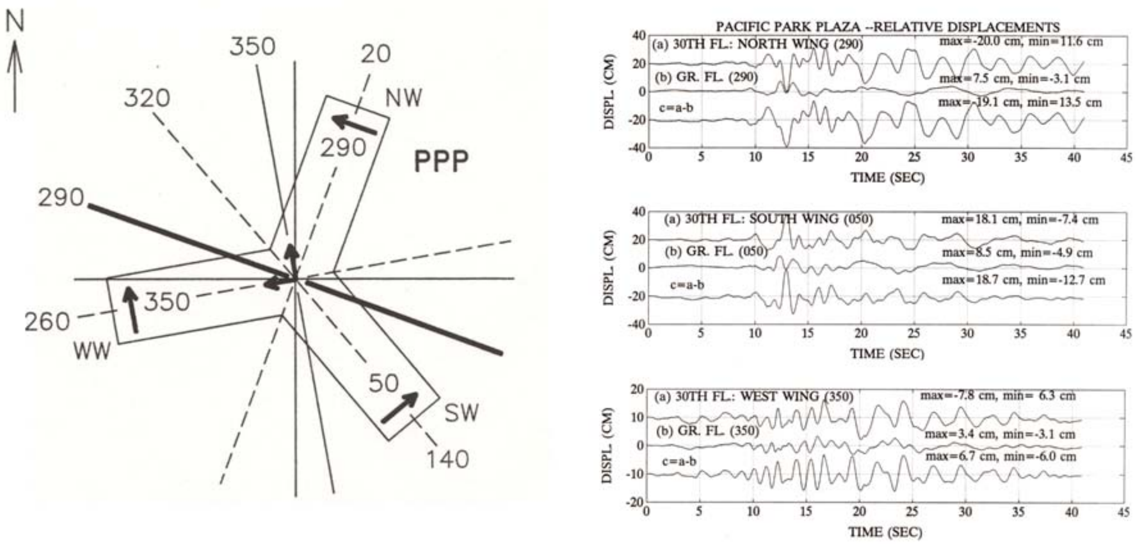

Figure 6. *Left: Orientation in plan of the building and of the accelerometers in the three wings. Heavy solid line exhibits the dominant direction of the ground motions affecting the building (Çelebi, 1992). Right: Relative displacements of the three wings exhibit different amplitudes during twisting of the wings with respect to the ground level.*

ROTATION AND ROCKING

State-of-the-art practice and analytical approaches require, when warranted, the structure-foundation system to be represented by mathematical models that include the influence of the sub-foundation media. In many cases, within a specific geotechnical environment, certain structures will respond differently than if built as a fixed based structure on a very stiff (e.g., rock) site. During many earthquakes, numerous structures exhibited rocking behavior. The alteration of vibrational characteristics of structures due to soil-structure interaction (SSI) behavior may have beneficial or adverse effects on performance of structures (Tarquis and Roesset, 1988; Kalkan and Grazier, 2007a). To date, the engineering community is not clear about the pros and cons of SSI.

Adverse effects of SSI during the 1985 Michoacan (Mexico) earthquake were addressed by Tarquis and Roesset (1988), who showed that, in the lakebed zone of Mexico City, 400 km away from the epicenter, fundamental periods of mid-rise buildings (5-15 stories) lengthened due to SSI. Thus, such buildings were negatively affected due to SSI because the lengthening of their fundamental periods placed them in a resonating environment close to the approximately 2-second resonant period of Mexico City lakebed.

On the other hand, under different circumstances, SSI may be beneficial because it produces an environment whereby the structure escapes the severity of shaking due to shifting of its fundamental frequency. Certainly, in a basin such as that of the Los Angeles area, SSI may cause either beneficial and detrimental effects in the response of structures. Thus, the identification of the circumstances under which SSI is beneficial or detrimental and the relevant controlling parameters is a necessity.

Rocking, a form of soil-structure interaction (SSI), occurs when a structural response involves contributions from rigid body rotation of the system, including its foundation, in addition to contributions from bending and shear deformations. In figure 7, the rocking contribution to the total response of the structure is represented as $h\theta_o$ (for small rotations, $\sin\theta_o \cong \theta_o$).

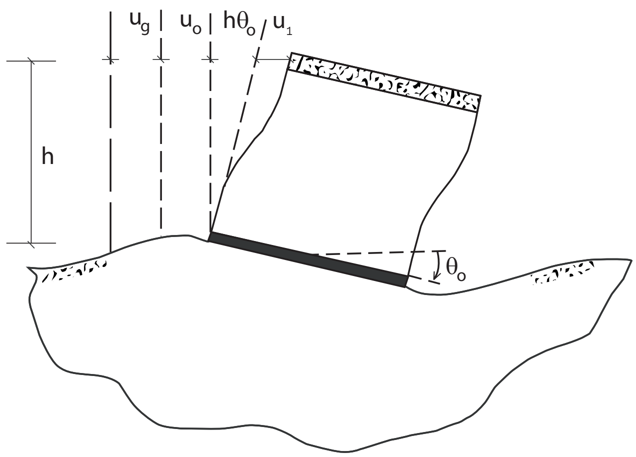

Figure 7. *Generic representation of rocking response of a single story structure to ground displacement, u_g.*

One of the best examples of rocking occurring during strong shaking is for the Transamerica Building (San Francisco, CA) during the 1989 Loma Prieta Earthquake. The Transamerica Building was designed according to code requirements in force in 1972; however, design evaluation was made using a site-specific design response spectrum with seismic forces that were higher than the code requirements (Çelebi, 1998). The pyramid-shaped, steel building is sixty stories, 257.3 m (844 ft) high and square in plan. At the ground level, the plan dimensions are 53 m x 53 m (174 ft x 174 ft). This plan starts reducing at the second floor to 44 m x 44 m (145 ft x 145 ft) at the 5th floor and then follows an exterior wall slope of 1 to 11 upwards. A perimeter truss system decorates and supports the building between the second and fifth floors. In addition to the exterior frame system, interior frames extend to the top of the structure with some of them ending at the 17th and 45th floors. The exterior pre-cast concrete panels are attached structurally to the exterior frames. The basement (three levels below the ground level) consists of a very rigid shear-wall box system. The foundation of the building consists of a 2.7 m (9 ft.) thick basemat without piles. The underlying soil media below the foundation consist, in general, of clays and dense sands. Below the ground level to a depth of 8 m (25 ft.), there are weak and compressible sand and rubble fill and recent bay deposits of sand and clay. Below 20 m (60 ft.), the sands are partially cemented. The bedrock is between 48-60 m (145--185 ft.) below the present street grade.

By an array of strong-motion instruments deployed by USGS in 1985, the response of the Transamerica Building was recorded during the October 17, 1989 Loma Prieta, CA earthquake [LPE] (M_s= 7.1), the epicenter of which is located 97 km from the building. This data set is very important as it reveals significant soil-structure interaction (SSI) effects associated with the earthquake response of the building. The array of instruments that recorded this effect are depicted in Figure 8, which shows a three-dimensional schematic of the building, the overall dimensions, the instrumentation scheme[2], and the recorded accelerations and displacements at selected locations within the building for the LPE. The instrumentation scheme was designed and implemented to study the response and associated dynamic characteristics of the building, including its translational, rocking and torsional motions. At the 21st, 5th and ground levels, three uniaxial accelerometers are deployed, two parallel to one another at the building West and East ends (building NS orientation is 351^o clockwise from true North) and the third with a nominal EW orientation (81^o clockwise from true North). These orientations are coincident with the orientations of the horizontal channels of the three SMA1's at the 49th, 29th and basement levels. The remaining four uniaxial accelerometers are deployed in the basement; three positioned vertically at three corners of the building, and one positioned horizontal, parallel to the nominal NS horizontal channel of the triaxial accelerograph in the basement. The senses of the orientations of the channels are also shown in Figure 8. In summary, there are parallel pairs of horizontal accelerometers in each of the 21st, 5th, ground, and basement levels, and another single accelerometer deployed orthogonal to each pair in the horizontal direction at the same levels. However, because the building is in a heavily built-up area of San Francisco, there is no appropriate location for a free-field array in the immediate vicinity of the building.

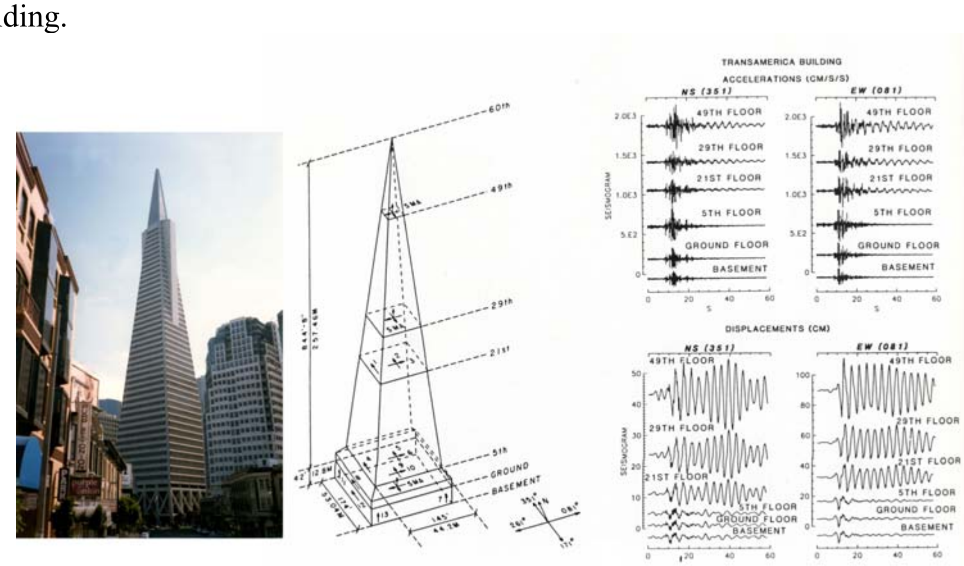

Figure 8. *Left: Photo of Transamerica Building, San Francisco, CA. Middle: Instrumentation schematic with arrows showing location and orientation of sensors. Right: The recorded accelerations and computed displacements for the 1989 Loma Prieta earthquake.*

[2] In 1998, instrumentation of Transamerica Building was upgraded by replacing triaxial units on 29th and 49th floors with uniaxial sensors similar to 21st floor. The instrumentation configuration is now updated to include a modern digital recording system and uniaxial accelerometers on the 29th and 49th floors, similar to the 21st floor.

From the records of the LPE, the dominant frequency (or period) for the fundamental mode is 0.28 Hz (3.6 seconds) in both the NS and EW directions, as extracted from the spectral analyses and system identification techniques. The ARX (acronym -- AR for autoregressive and X for extra input) model based on the Least Squares method for single input-single output coded in commercially available system identification software (The Mathworks, 1988) is used in system identification analyses performed herein (Ljung, 1987). Simply stated, the input is the basement or ground floor motion and the output is the roof level motion or one of the levels where the structural response is recorded. The damping ratios are extracted with the procedures outlined by Ghanem and Shinozuka (1995), and Shinozuka and Ghanem (1995). Other frequencies are 0.5, 1.2, 1.5 and 1.8 Hz for the EW direction and 1.0, 1.35, 2.0, and 2.6 Hz for the NS direction. Sample results from the application of system identification technique for the Transamerica Building records are shown in Figure 9. In this application, motions at the 49^{th} floor are used as output and those at the basemat as input. The match between the observed and calculated response is excellent, as evidenced by comparison of the calculated and observed amplitude spectra of the responses at the 49^{th} floor. The critical viscous damping ratios extracted from the system identification analyses corresponding to the 0.28 Hz first mode frequency are 4.9 % (NS) and 2.2 % (EW) (Çelebi and Safak, 1991; Çelebi, 1998). The analyses of the records showed that there is no significant torsional motion, as evidenced by analyses of the differences in the parallel accelerations and displacements (Çelebi and Safak, 1991).

Additional detailed analyses results will not be repeated herein. However, to demonstrate the likely presence of rocking, a significant contributor to SSI, the horizontal motions recorded at the 21^{st} floor and the vertical motions recorded at the basemat are used. Shown in Figure 10 are the coherency, phase angle and cross-spectrum plots for the NS (351^o) direction of a pair of horizontal acceleration on the 21^{st} floor and vertical acceleration of the basemat. It is observed that the rocking motion occurs at 0.5 seconds (or 2.0 Hz) in the NS (351^o) direction, and that at this frequency, the horizontal motion at the 21^{st} floor and the vertical motion in the basement are coherent and are in phase. What this demonstrates is that there is significant and coherently identifiable rocking motion of this particular building that manifests itself in altering the dynamic characteristics and the response (e.g., lengthened period [shortened frequency] of 3.57 seconds [0.28 Hz] of the building as compared to those from the low-amplitude tests with 2.94 s [0.34 Hz]) which did not have rocking mode.

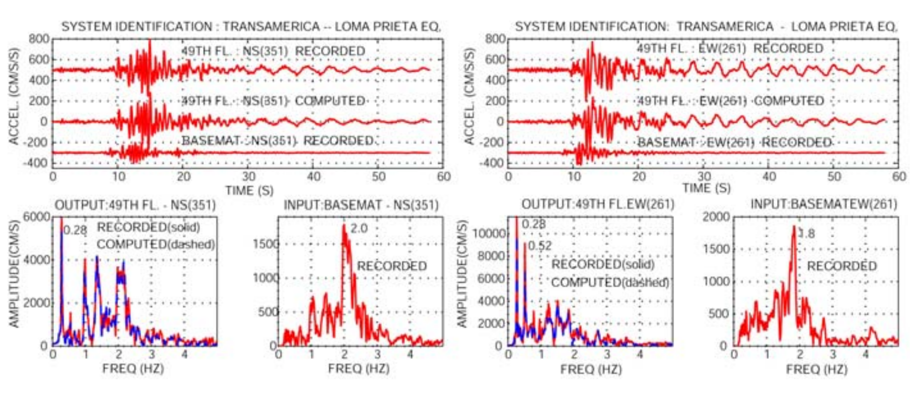

Figure 9. *System identification applied to LPE motions recorded at 49^{th} floor (output) and the basemat (input) for the Transamerica building. Identified frequency is 0.28 Hz.*

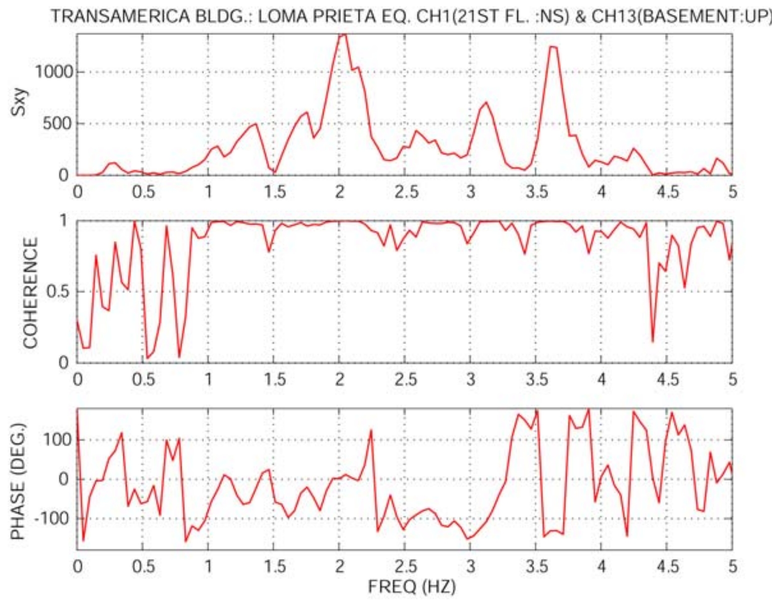

Figure 10. *Cross-spectrum, coherency and phase angle plots of pairs of motions (NS at 21st floor and vertical in the basement) indicate rocking at 2 Hz (0.5 sec).*

ROTATION OF BASEMAT AND BASEMENT WALLS

Determination of rotation of basemat of a building around a horizontal axis is important as it indicates degrees of SSI effects. In general, if the basemat is rigid enough, the rotation about a horizontal axis is indicative of rocking. In current monitoring programs, measurement of rocking is achieved by deploying several vertical accelerometers in corners of the basemat. With known distances between them, rocking, if any, can be assessed. Other measurements of translational motion can be used to qualitatively assess rocking. In this section, the aim is to present measurements of rotation of basemat and basement walls. In practice, motions at basemat and ground level of a building with several levels of basement floors are considered similar. Engineers usually ignore possible soil-structure interaction that takes place in basements. As shown in Figure 11, for the Transamerica Building (Figure 8), differential displacements indicative of rotations are computed for the basement walls using ground level and basemat horizontal motions (Figures 11a and b), and the differential vertical displacements (Figure 11c)of the basemat are computed using vertical motions at the corners of the basemat. There is an order of amplitude difference between the rotations of the basement walls and basemat as seen in the comparative Figure 11d. This indicates effect of dynamic earth pressures on the basement walls and soil-structure interaction (Soydemir and Çelebi, 1992). Similar results were obtained for the Embarcadero Building basement walls and basemat as evaluated by displacements computed from acceleration recorded from uniaxial translational accelerometers deployed throughout the building (Çelebi, 1993, 1998).

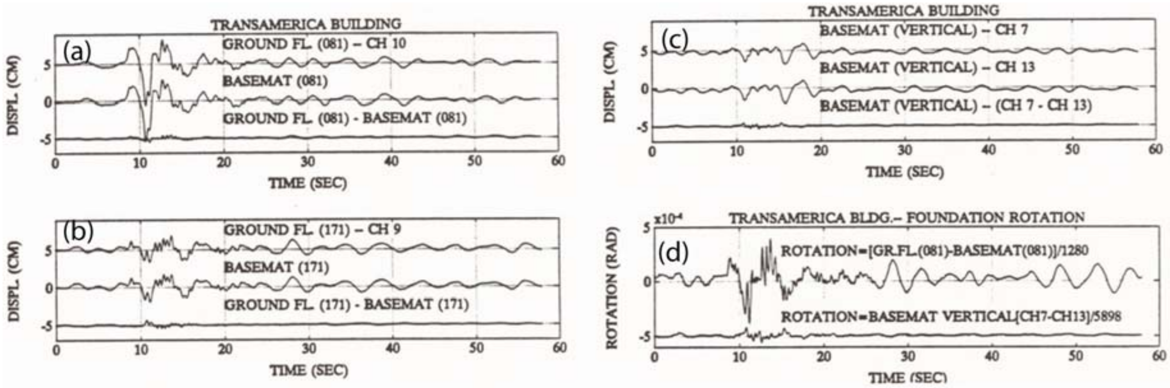

Figure 11. *Displacements and rotations of basemat and basement walls at Transamerica Building. (Note: Rotations are computed from displacements. (a,b) show differential displacements indicative of rotations of the basement walls using ground level and basemat horizontal motions. (c)shows the differential vertical displacements of the basemat computed using vertical motions at the corners of the basemat. (d)compares rotation of the basement wall with that of the basemat. Displacements are computed by double integration of accelerations recorded by uniaxial translational accelerometers)*.

The same phenomena with similar order of magnitude differences in amplitudes of rotation of basement walls and basemats are observed from the recorded responses of the Embarcadero Building during the 1989 Loma Prieta, CA earthquake (Figures 12 and 13).

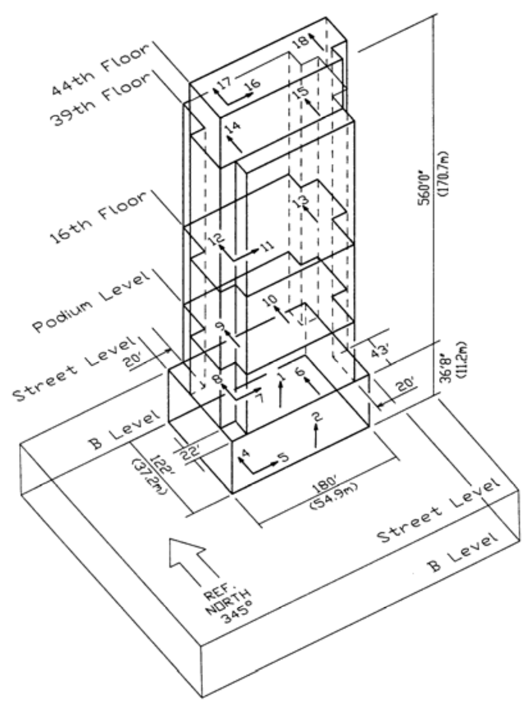

Figure 12. *Schematics and instrumentation scheme of Embarcadero Building in San Francisco, CA*.

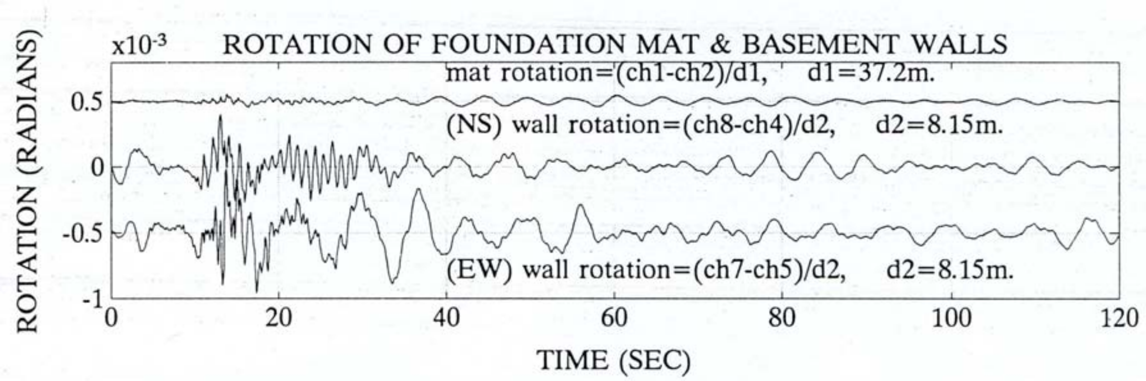

Figure 13. *Rotations of basement walls and mat foundation computed from displacements derived from accelerations recorded with uniaxial translational accelerometers of the Embarcadero Building. (Note: As in Transamerica Building, there is an order of magnitude of differences in amplitudes of rotations of basemat compared with basement walls (Çelebi, 1998)).*

CONCLUSIONS

It is shown in this paper that engineers consider, assess, measure and incorporate in their designs and analyses the significant behavioural actions caused by rotations of sections, members, joints and a structure as a whole. It is also shown that there are many variations of actions for which rotations are important variables. For bending, torsion, twisting, rocking and other important response actions, examples provided depict rotations computed from actual measurements of displacements computed from accelerations recorded with uniaxial translational accelerometers.

In the vast majority of instrumented structures, to date, rotational sensors have not been deployed. Yet it is still possible to extract the rotational motions from differentials of parallel, uni-axial translational motions acquired by sensors strategically deployed throughout a structure for this purpose.

It is possible that in the future, if feasible, rotational sensors can be used to directly measure rotations and complement the use of along with uniaxial translational sensors for this purpose.

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