Seismic Behavior and Modeling of Anchored Nonstructural Components Considering the Influence of Cyclic Cracks

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by

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The Dissertation of Derrick Andrew Watkins is approved, and it is acceptable in quality and form for publication on microfilm:

Chair

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NOTATIONS

In general, notations for anchor terminology have been taken from the American Concrete Institute (ACI). Anchor terminology from European Technical Standards (ETAG, DIBt) is somewhat different. This project is international and consideration has been given to unify notation where possible. One example is crack width; the international community uses crack width in millimeters, typically 0.5 mm [0.02 inch] and 0.8 mm [0.03 inch] as standard crack widths for testing. Throughout this report, crack width is referred to in millimeters. Duals units have been provided in figures where possible. Another deviation from the ACI standard is the variable for axial stiffness, where $K$ is used instead of $\beta$. The following section defines the indices, subscripts, acronyms, and variables used in this dissertation.

Indices

$i$  Time, as in discrete data sampled at time $i$
$j$  Anchor location or data channel
$k$  Cycle number, number of cycles
$n$  Test number, or total number of tests

Subscripts

$d$  Duration, length of time
$cr$  Cracked concrete, crack, or crack cycling
$uncr$  Uncracked concrete
$m$  Arithmetic mean
$max$  Maximum
$min$  Minimum
$u$  Ultimate, typically same meaning as maximum, but associated with anchor load
$ref$  Reference
## Acronyms/Abbreviations

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<thead>
<tr>
<th>Abbreviation</th>
<th>Meaning</th>
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<tr>
<td>C</td>
<td>Concrete breakout cone failure</td>
</tr>
<tr>
<td>CC</td>
<td>Cracked Concrete</td>
</tr>
<tr>
<td>CM</td>
<td>Center of Mass</td>
</tr>
<tr>
<td>CCILR</td>
<td>Cyclic Crack Inertial Loading Rig</td>
</tr>
<tr>
<td>DAQ</td>
<td>Data Acquisition</td>
</tr>
<tr>
<td>D/C</td>
<td>Demand to Capacity ratio</td>
</tr>
<tr>
<td>FFT</td>
<td>Fast Fourier Transform</td>
</tr>
<tr>
<td>FM</td>
<td>Floor Motion</td>
</tr>
<tr>
<td>FRF</td>
<td>Frequency Response Function</td>
</tr>
<tr>
<td>kip</td>
<td>Kilo pounds (1 kip = 1000 lbs)</td>
</tr>
<tr>
<td>ksi</td>
<td>Kips per square inch</td>
</tr>
<tr>
<td>LW</td>
<td>Load Washer</td>
</tr>
<tr>
<td>NCS</td>
<td>Nonstructural Components and Systems</td>
</tr>
<tr>
<td>Po</td>
<td>Pullout failure (sleeve pulls out of hole)</td>
</tr>
<tr>
<td>psi</td>
<td>Pounds per square inch</td>
</tr>
<tr>
<td>PSD</td>
<td>Power Spectral Density</td>
</tr>
<tr>
<td>Pt</td>
<td>Pull-through failure (sleeve stays in hole)</td>
</tr>
<tr>
<td>S</td>
<td>Steel failure</td>
</tr>
<tr>
<td>Sp</td>
<td>Splitting failure</td>
</tr>
<tr>
<td>SAMU</td>
<td>Shear and Axial Measurement Unit</td>
</tr>
<tr>
<td>SISO</td>
<td>Single Input Single Output</td>
</tr>
<tr>
<td>SDOF</td>
<td>Single Degree of Freedom</td>
</tr>
<tr>
<td>SDSU</td>
<td>San Diego State University</td>
</tr>
<tr>
<td>UC</td>
<td>Uncracked Concrete</td>
</tr>
<tr>
<td>UCSD</td>
<td>University of California San Diego</td>
</tr>
<tr>
<td>WALLE</td>
<td>Weighted Anchor Loading Laboratory Equipment</td>
</tr>
</tbody>
</table>
Variables

Accl Acceleration, g

$A_{accl_{WALLE,CM}}$ Acceleration (absolute) at the center of mass of WALLE, in

$A_g$ Gross section area, in$^2$

$a$ WALLE position ratio, unitless

$a_{cm}$ Acceleration (absolute) at the center of mass, g

$a_f$ Floor absolute acceleration, g

$a_g$ Ground acceleration, g

$d_{NCS}$ NCS absolute acceleration at center of mass, g

$\alpha_{cc}$ Ratio of constant load to mean ultimate load, unitless

$\alpha_i$ Pivot point location parameter, unitless

$c$ Distance from extreme compression fiber to neutral axis at joint, in

$C_c$ Resultant compression force in concrete, kip

$c_c$ Distance from extreme unconfined fiber to extreme confined fiber, in

$CV$ Coefficient of variation, unitized standard deviation, unitless

$D_{Disp_{WALLE,CM}}$ Displacement (absolute) at the center of mass of WALLE, in

$d$ Diameter of anchor, typically the thread rod diameter, in

$d_{cone}$ Diameter of concrete breakout cone (measured at the surface of the concrete), in

$d_{cut}$ Diameter of cutting part of a drill bit, largest diameter of cutting blades, in

$d_{hole}$ Depth of drill hole, in

$\delta$ Axial displacement of anchor, in

$\delta_{Nmax}$ Displacement at maximum load, in

$\delta_{pp}$ Post-peak displacement at load equal to 85% of ultimate on descending branch, in

$\delta_u$ Displacement at ultimate load, in

$\delta_{um}$ Mean displacement at ultimate load, from a series of tests, in

$\delta_{um,cr}$ Mean displacement at ultimate load, in cracked concrete, in
<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\delta_{um,uncr}$</td>
<td>Mean displacement at ultimate load, in uncracked concrete, in</td>
</tr>
<tr>
<td>$\delta_i$</td>
<td>Initial displacement at load equal to 50% of ultimate on ascending branch, in</td>
</tr>
<tr>
<td>$d_b$</td>
<td>Diameter of reinforcement bar, in</td>
</tr>
<tr>
<td>$\Delta_{CM}$</td>
<td>Displacement at center of mass, in</td>
</tr>
<tr>
<td>$\Delta_w$</td>
<td>Crack opening, additional crack width after hairline crack, in</td>
</tr>
<tr>
<td>$\varepsilon_y$</td>
<td>Yield tensile strain in steel reinforcement</td>
</tr>
<tr>
<td>$E_c$</td>
<td>Modulus of elasticity of concrete, ksi</td>
</tr>
<tr>
<td>$E_s$</td>
<td>Modulus of elasticity of steel reinforcement, ksi</td>
</tr>
<tr>
<td>$f$</td>
<td>Frequency of vibration, Hz</td>
</tr>
<tr>
<td>$f_c$</td>
<td>Compressive stress in concrete, ksi</td>
</tr>
<tr>
<td>$f'_c$</td>
<td>Unconfined concrete compressive strength, ksi</td>
</tr>
<tr>
<td>$f_s$</td>
<td>Equivalent static force, kip</td>
</tr>
<tr>
<td>$f_u$</td>
<td>Tensile strength of steel reinforcement, ksi</td>
</tr>
<tr>
<td>$f_y$</td>
<td>Tensile yield stress in energy dissipating reinforcement, ksi</td>
</tr>
<tr>
<td>$F$</td>
<td>Force, kip</td>
</tr>
<tr>
<td>$F_{y1}$</td>
<td>Tension yield force, kip</td>
</tr>
<tr>
<td>$FS$</td>
<td>Factor of safety, ratio of capacity / demand, unitless</td>
</tr>
<tr>
<td>$F_p$</td>
<td>Seismic component force, kip</td>
</tr>
<tr>
<td>$h$</td>
<td>Height of the roof of a building relative to ground, ft</td>
</tr>
<tr>
<td>$h_{3,4,etc}$</td>
<td>Height to accelerometer 3, 4, etc (measured vertically from the WALLE base), in</td>
</tr>
<tr>
<td>$h_{ef}$</td>
<td>Effective embedment depth, in</td>
</tr>
<tr>
<td>$h_{min}$</td>
<td>Minimum slab thickness, in</td>
</tr>
<tr>
<td>$h_{proj}$</td>
<td>Height of anchor projected above the surface of concrete, in</td>
</tr>
<tr>
<td>$I_A$</td>
<td>Arias intensity of acceleration history, $g^2$</td>
</tr>
<tr>
<td>$I_{\lambda,cr}$</td>
<td>Arias intensity of normalized crack history, unitless</td>
</tr>
<tr>
<td>$I_g$</td>
<td>Gross moment of inertia of concrete section, $in^4$</td>
</tr>
<tr>
<td>$k$</td>
<td>Effectiveness factor for concrete tension, $in^{-0.5}lb^{0.5}$</td>
</tr>
</tbody>
</table>
Stiffness, kip/in

Effectiveness factor for concrete tension in cracked concrete, $in^{-0.5} lb^{0.5}$

Effectiveness factor for concrete tension in uncracked concrete, $in^{-0.5} lb^{0.5}$

Stiffness, kip/in

Axial secant stiffness at 50% of $N_u$, ascending branch, kip/in

Axial secant stiffness at 50% of $N_u$, ascending branch, kip/in

Axial secant stiffness at 80% of $N_u$, descending branch, kip/in

Axial secant stiffness at 85% of $N_u$, descending branch, kip/in

Axial secant stiffness at ultimate load, $N_u$, kip/in

Length, length of slab, in

Curvature normalized by yield curvature, unitless

Cracked strength reduction factor, ratio of $N_{um,cr}/N_{um,uncr}$, unitless

Ratio of anchor displacement in cyclic cracks to mean monotonic disp., unitless

Moment, kip-in

Mass, lb-sec$^2$/in

Mean, average value, in units of the quantity under evaluation

Test number or total number of tests

Axial load in anchor ($+N$ = tension), kip

Characteristic strength, 95% probability of exceedance with 90% confidence, kip

Axial load in anchor at time “i” at anchor location “j”, kip

Maximum axial load in anchor, kip

Maximum axial load in anchor during load cycle “i”, kip

Axial load in anchor at time “i” at anchor location “j”, kip

Normalized axial load in anchor, unitless

Normalized axial load in anchor at instant of time “i”, unitless

Ultimate axial load capacity of anchor, peak/maximum value, kip

Mean ultimate axial load capacity of anchor, kip

Mean ultimate axial load capacity of anchor, quasi-static loading, kip
\( N_{um,cr} \) Mean ultimate axial load capacity of anchor in cracked concrete, kip
\( N_{um,uncr} \) Mean ultimate axial load capacity of anchor in uncracked concrete, kip
\( N_b \) Concrete breakout strength, kip
\( N_p \) Pullout/pull-through strength, kip
\( N_{pp} \) Post peak anchor tension capacity (on descending branch of \( N-\delta \) curve), kip
\( N_{var} \) Percentage of load carried by an anchor
PIA Peak input acceleration, g
PID Peak input displacement, in
PWA Peak WALLE acceleration, g
PWD Peak WALLE displacement, in
\( \phi \) Material strength reduction factor applied to resistance for design
\( \phi(t) \) Curvature time history
\( \phi_y \) Effective yield curvature, radians/inch
\( \Psi_{c,N} \) Factor for cracked tensile strength modification, ratio \( N_{um,uncr} / N_{um,cr} \), unitless
\( R_{max} \) Radius of concrete breakout cone, maximum, in
\( R_{min} \) Radius of concrete breakout cone, minimum, in
\( R_p \) Component response modification factor
\( s_{min} \) Minimum spacing, in
\( S_a \) Spectral acceleration, g
\( S_{DS} \) 5 percent damped spectral response acceleration parameter at short periods, g
\( \sigma \) Standard deviation, in units of the quantity under evaluation
\( t_d \) Effective duration of strong motion, seconds
\( t_{d,cr} \) Effective duration of strong crack cycling, seconds
\( T_n \) Natural period of structure or NCS, seconds
\( T_1 \) First natural period of structure or NCS, seconds
\( T_{inst} \) Installation torque, manufactures recommended value, in-lb
\( T_{test} \) Test torque, during test, typically 50% of manufactures recommended value, in-lb
\( T_p \) Predominant period of acceleration, seconds
\( T_{p,cr} \) Predominant period of crack cycling, seconds
$T_{p,N}$  Predominant period of anchor loading, seconds
$T^*$  Period ratio, unitless
$T_{WALLE}$  Natural period of WALLE, seconds
$T_{xy}$  Transfer function of signals x and y, unitless
$\theta$  Angle of rotation, degrees
$u$  Displacement of an SDOF relative to its base, in
$u_t$  Absolute displacement, in
$u_g$  Displacement of the ground, in
$\ddot{u}$  Velocity of the mass relative to its base, in/second
$\dddot{u}$  Acceleration of the mass relative to the base, g
$\dddot{u}_g$  Acceleration of the ground, g
$\dddot{u}_r$  Absolute acceleration of the mass, g
$V$  Shear load in anchor, kip
$V_{\text{anchors}}$  Total portion of base shear resisted by all anchors, kip
$V_n$  Nominal shear capacity, kip
$V_u$  Factored shear load in anchor, kip
$\omega_n$  Circular natural frequency, radian/sec.
$w$  Crack width, mm
$w(t)$  Crack width time history
$w_{\text{hairline}}$  Hairline crack width, mm
$w_i$  Crack width at instant of time “i”, mm
$w_{\text{max}}$  Maximum crack width, mm
$w^*$  Crack width normalized by $w_{\text{max}}$ per anchor, unitless
$w^*_i$  Crack width normalized by $w_{\text{max}}$ per anchor at instant of time “i”, unitless
$w/c$  Volumetric water to cement ratio, unitless
$W_p$  Weight of a nonstructural component (operating weight), kip
$z$  Height NCS point of attachment in a building relative to the ground, ft
$\zeta$  Damping, expressed as a percentage of critical damping, unitless
ACKNOWLEDGEMENTS

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Chapter 2 contains material that was published in the following journal papers in which the author of this dissertation was either the primary or a co-author:


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ABSTRACT OF THE DISSERTATION

Seismic Behavior and Modeling of Anchored Nonstructural Components Considering the Influence of Cyclic Cracks

by

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Doctor of Philosophy in Structural Engineering

University of California, San Diego, 2011

Professor Tara Hutchinson, Chair

During an earthquake, reinforced concrete members in a building will suffer cracking that oscillates as the building dynamically deforms. Equipment that services the building, such as mechanical and electrical items, is anchored to these components, and therefore will be subjected to this dynamic environment. Despite understanding this practical loading situation, as well as recognizing that anchor load capacity is significantly reduced when an anchor is embedded in cracked concrete, there remains a gap in knowledge regarding the effect of anchorage behavior on nonstructural component response. In particular, the effect of dynamic
cyclic cracking coupled with inertia-generated tension load cycling on the anchor and component response has not been studied to date.

A new methodology involving experimental equipment and simulation tools is developed for investigating the seismic behavior of anchored nonstructural components and systems that accounts for the effects of simultaneous anchor tension load cycling and crack cycling on anchor behavior and anchored component response. To support the experimental ingredient of this work, a Cyclic Cracked Inertial Loading Rig (CCILR), Weighted Anchor Loading Laboratory Equipment (WALLE) system, and cracked concrete slabs, are designed and fabricated. Mounting the CCILR and WALLE onto a shake table results in a system that is able to simulate a concrete beam or slab from a building supporting an anchored nonstructural component. System-level shake table tests are conducted on floor mounted model nonstructural components anchored in cyclic cracks using epoxy, expansion, drop-in and undercut anchors to study the effect of a range of anchor types.

A nonlinear, lumped hysteresis anchor model is implemented and used to simulate anchor load-displacement response for tension load cycling dominant applications. The anchor model is calibrated against single anchor tests and subsequently extended for use in a system model of the anchored nonstructural components for predicting maximum system response.

It was determined that the load-displacement behavior of the anchorage, in particular, the ultimate displacement capacity of the anchor, plays an important role in the seismic response of tension load cycling dominated floor mounted nonstructural components. The experimental results also support the current code design philosophy for anchors, which specifies that either the anchor or the attachment should be ductile, or the anchor should be designed for a multiple of the expected load demand.
Chapter 1 Introduction

1.1 Motivation

This research investigated the seismic behavior of anchored Nonstructural Components and Systems (NCSs). NCS are those systems within a structure that are not intended to contribute to the structure's load-bearing system. Nevertheless, these systems will be subject to the dynamic environment of the building, and thus may be subject to damage under even moderate earthquake loading. Broadly, NCSs are categorized as either: (i) architectural, (ii) mechanical and electrical or (iii) building contents. Although NCSs are not part of the main structural system, they are required to resist loads and displacements to maintain attachment to the structure for life-safety. In addition, they must maintain function following a seismic event when required to support critical public services, such as hospitals, fire and police stations, power stations, nuclear facilities, water supply and water treatment plants.

In many cases, NCSs are anchored to the primary structure by means of cast-in-place or post installed anchors. Figure 1(a) shows a pump package anchored to a concrete floor slab in a hospital. Figure 1(b) shows a detail of the anchorage location for this equipment. In this case, the post-installed anchor is fastened into concrete, which has a crack intersecting the anchor location. Many mechanical post-installed anchors are expanded against the drilled hole during installation to develop frictional resistance to tension loads. The expansion forces generated during installation, coupled with the stress concentration induced by the drilled hole can cause a crack to form during installation. Research has shown that anchors located in cracks experience a reduction in both axial stiffness and axial strength (e.g. Eligehausen et al., 2006).
Earthquakes generate displacements and inertial forces in buildings, herein referred to as primary structures. Primary structures filter the ground motion input and result in floor accelerations, which serve as input to the NCSs, also called secondary systems (Figure 1.2a). The NCSs transfer their inertial loading to the anchors, resulting in dynamic tension and shear loading on the anchors. For seismically loaded anchored NCSs, the magnitude of the anchor loads and number of load cycles is highly dependent on the dynamic characteristics of the primary structure and in particular the NCS dynamic properties (e.g. Tang and Deans, 1983; Ammann, 1992).

An earthquake also causes dynamic strains in the primary structure, which in turn results in cracks that cycle opened and closed in concrete structural members. Active crack closure occurs because of reversal in the direction of load demands caused by the cyclic nature of the seismic input. Depending on the dynamic characteristics of the primary and secondary structure, cracks may be partially open, fully open, or closed coincident with peak loading in the anchors. Crack width in concrete members has been the subject of a large amount of
research (e.g. Broms, 1965; Broms and Lutz, 1965; Gergely and Lutz, 1968; Nawy 1968; Rehm and Martin 1968; Oh and Kang 1987; Martin and Swartzkopf 1980; Bazant and Oh, 1983; Beeby, 1990; Frosch 1999; Eurocode Pt 2, 2002; Hoehler, 2006; Wood et al., 2009; Marzouk et al., 2010). Typical design codes require that anchorage be located outside the plastic hinge zone. Therefore, the maximum flexural crack width occurs at insipient yielding of the reinforcement (Hoehler, 2006). In this context one may relate $w_{max}$ to beam or slab yield curvature. Wood et al. (2009) for example, finds $w_{max}$, as associated with beam yielding, to range from 0.4 to 0.6 mm (0.016 to 0.024 in.), while Hoehler (2006) found this range to be from 0.4 to 1.0 mm (0.016 to 0.04 in.).

Under earthquake loading, the crack time history and general cracking characteristics (distribution and amplitude) in concrete members will be dictated by the deformation pattern of the primary structure. Simultaneously, anchor forces will develop in response to the inertial response of the component. The behavior of the anchors in cracked concrete (stiffness, deformation, and strength) in turn affects the response of the anchored component. This complex interaction is difficult to mimic in a laboratory setting, absent a full-scale model building or segment of a building. To realize such conditions, a methodology is needed to simulate simultaneously the boundary and loading conditions of the anchored-NCS subsystem. A sketch of the needed simulation methodology and experimental framework is shown in Figure 1.2, where the numerically developed floor acceleration and crack width histories are used as input to the experimental simulation. In this dissertation, the experimental component of this methodology is developed and exercised considering floor-mounted model NCSs in which the anchorage is tension load dominated. Experimental results provide input to calibrate a simplified numerical model. The model adopts a nonlinear lumped hysteresis spring to capture anchor behavior. The anchor model is calibrated against single anchor tests.
and subsequently extended for use in a system model of the anchored NCS for predicting maximum system response.

Figure 1.2 (a) Analytical simulation of actions on structures, non-structural components and anchors and (b) experimental simulation of primary and secondary system behavior

1.2 Problem Statement

The seismic performance of anchors in cracked concrete has historically been investigated under load and boundary conditions that do not fully represent the dynamic environment encountered when a structure is subjected to earthquake loads. Few investigations have considered the complex interplay between anchor behavior and the components they connect. This research presents a new methodology and associated experimental setup for studying the seismic behavior of anchored components. This work aims at capturing the salient features of boundary and loading conditions of anchored components within building systems, whereby the component is subjected to seismic-induced...
loading, while the anchor is embedded in cracks that cycle opened and closed. The objective of this research is to study this issue experimentally and numerically and to contribute to development of improved design methods for anchored NCSs.

1.3 State-of-the-Art

1.3.1 Anchorage Types and Failure Mechanisms

Anchors may be placed in two categories based on the time of installation. If the anchor is installed into a drilled hole after concrete has hardened, it is called a post-installed anchor (Figure 1.3a). If the anchor is placed into the formwork before fresh concrete is placed, it is called a cast-in-place anchor (Figure 1.3b). Eligehausen et al. (2006) further classifies anchors according to the tension load transfer mechanism from the anchor to the concrete base material. The most common tension load transfer mechanisms are mechanical interlock, friction, and bond (Figure 1.4). Different anchor types make use of the transfer mechanisms to fasten to the base concrete. There are a variety of anchor types available such as undercut anchors, torque-controlled expansion anchors, sleeve anchors, adhesive/bonded anchors, displacement-controlled expansion anchor, also called drop-in anchors, and screw anchors (Figure 1.5).
Figure 1.3 Types of anchors (ACI committee 318, 2008)

(a) Post-installed anchors

(b) Cast-in-place anchors

Figure 1.4 Tension load transfer mechanisms for anchors (Eligehausen et al. 2006)
For anchor loading in tension, five distinct failure modes have been identified: (a) steel failure, (b) pull-out, (c) pull-through, (d) concrete cone failure, and (e) splitting failure (Figure 1.6). Steel failure occurs when the strength of the concrete is higher than the steel element and the steel yields and fractures. Pullout occurs when the anchor has lost a significant amount of frictional resistance or bearing strength. Pullout results in an entire anchor pulling out of the drilled hole, without a significant concrete breakout cone occurring. Pull-through type failure occurs when the anchor pulls through the expansion element, typically an expansion wedge or sleeve. Concrete cone failure initiates at the head of the anchor and propagates upward in the concrete in a radial circular cone at approximately 35 degrees. Concrete cone failure strength is related to the effective embedment depth and
concrete properties such as tensile strength and fracture energy (Eligehausen et al., 2006). The concrete strength, maximum aggregate size, and the type of aggregate influence the fracture energy. Fracture energy is inherently accounted for when performing physical testing to determine the concrete breakout strength. Splitting failure occurs when the anchor expansion forces exceed the tensile strength of the surrounding concrete causing a crack to split open.

Figure 1.6 Anchor failure modes (adapted from ACI 318, 2008)

1.3.2 Formation of Cracks at Anchor Locations

One question that must be asked is “if a crack forms in a concrete slab, how likely is it that it will run directly through the location of an anchor?” To answer this question, Lotze (1987) performed a study on a reinforced concrete beam subjected to uniform moment (Figure 1.7). Their objective was to see if the crack pattern would follow the spacing of the transverse reinforcement or if the cracks would propagate to anchor locations. Several stress
discontinuities were studied; externally loaded anchors, anchors set, or prestressed, but not externally loaded, and drilled holes without anchors. In all cases, the holes for the anchors acted as a stress concentration that attracted the cracks. This and other studies indicate that it is reasonable, and conservative, to assume that the crack will directly intersect the anchor locations.

Figure 1.7 Crack propagation to anchor locations (Lotze, 1987)

1.3.3 Anchorage Behavior in Cracked Concrete

Previous research has shown that anchor behavior in cracked concrete is significantly different from its behavior in uncracked concrete (e.g. Cannon, 1981; Rehm et al., 1968 & 1982; Eligehausen and Balogh, 1995; EligehAUSEN et al., 2006; Hoehler, 2006). When located in cracks, the anchors experience a reduction in strength and stiffness and in some cases a
change in failure mode (Figure 1.8a). In general, as crack width increases, the anchor strength and stiffness decrease.

Figure 1.8 Effect of cracking on (a) tension load-displacement curves for a torque-controlled expansion anchor and (b) strength reduction factors in cracked concrete for (left) undercut anchors and headed studs, (middle) expansion anchors, and (right) adhesive anchors (from Hoehler 2006; Eligehausen and Balogh 1995)
Tests on numerous anchors loaded in tension in static cracks show an average strength reduction of 30% at a static crack width of 0.01 in (0.3 mm) (e.g. Eligehausen and Balogh, 1995; Eligehausen et al., 2006). To quantify the strength reduction, the tension strength reduction factor (Λ) is introduced, which is defined as the ratio $\frac{N_{um,cr}}{N_{um,uncr}}$ where $N_{um,cr}$ is the mean ultimate tension strength of anchor in cracked concrete and $N_{um,uncr}$ is the mean ultimate tension strength of an anchor in uncracked concrete. As shown in Figure 1.8b, the tension strength reduction factor is sensitive to anchor type (and hence failure mechanism).

### 1.4 Summary of Previous Research

The behavior of anchors in concrete has been determined experimentally assuming one or several of the following: (i) uncracked concrete, (ii) static cracks opened to a prescribed width coupled with static monotonic anchor loading (e.g. Cannon, 1981; Eligehausen and Balogh, 1995; Silva, 2001; and ACI Committee 355, 2007), (iii) static cracks opened to a prescribed width coupled with dynamic rate anchor loading (e.g. Rodriguez et al. 2001; and Hoehler et al. 2011a); (iv) static cracks coupled with cyclic anchor loading (e.g. SEAOSC, 1997; DIBt 1998; , Eligehausen, et al., 2004; Hoehler, 2006; Hoehler and Eligehausen, 2008b; AC193, 2009; and AC308, 2009); (v) static cracks coupled with inertial seismic loading (e.g. Rieder 2005; Sippel and Rieder, 2007; Rieder et al., 2008), and (vi) cycled cracks coupled with static loading (e.g. Rehm and Lehmann 1982; Furche 1987 and 1988; Hoehler, 2006; and Hoehler and Eligehausen 2008a). These approaches simplify the conditions that exist when a structure is subjected to an earthquake. Nonetheless, they support the current state of understanding and therefore will be reviewed in the following sections.
1.4.1 Static Crack Width with Anchor Loading Cycled

Sipple and Rieder (2007)

Sipple and Rieder (2007) performed tri-axial shake table tests on anchors in cracked concrete with a constant crack width of 0.06 inches (1.5mm). To the author’s knowledge, these are the first known tests where inertial loading was applied to anchors to generate combined shear and tension. Three types of post-installed anchors were investigated namely: undercut, expansion, and bonded expansion anchors. An artificially generated broad-band random acceleration history nominally 30 seconds in duration was used as the input to the shake table (IEEE 693, 1997). The authors conclude that the failure mode of the anchor depends on the anchor type. The undercut and expansion anchors failed by concrete surface spalling and bending of the bolt. The bonded expansion anchors pulled out fully from the concrete. The authors also suggest that large axial deformability of the anchor has a detrimental effect on anchor and system performance because the gap that forms between the concrete and the test fixture causes a pronounced “hammer effect.”

Hoehler and Eligehausen (2008b)

Hoehler and Eligehausen (2008b) performed experiments to investigate the failure mechanisms associate with tension load cycling at near-ultimate load levels when anchors are located in cracked concrete. The cracks were held at a constant crack width of 0.03 inches (0.8 mm). Post-installed anchor types investigated include sleeve anchors, expansion anchors, screw anchors, and modified expansion anchors. Two load cycling protocols were used: (1) constant load cycling to 50% and 90% of the mean ultimate static tension strength and (2) stepwise increasing load cycling to 15%, 30%, 45%, 60%, 75% and 90% of the mean ultimate static tension strength. Test results show that the load-displacement behavior for load cycling
closely follows the monotonic mean backbone load-displacement curve. The authors also conclude that the load cycling frequency (0.5Hz and 5Hz) did not negatively affect the load-displacement results or the residual anchor strength after load cycling for any of the failure modes. Results show a slight increase in ultimate load for the load cycling case with $N_{um}/N_{um\text{(static)}}$ ratios ranging from 1.03 (expansion anchor) to 1.28 (screw anchor) where $N_{um}$ is the mean ultimate strength and $N_{um\text{(static)}}$ is the mean ultimate strength when static monotonic load is applied. The authors conclude that anchors failing by concrete breakout performed unexpectedly well under tension load cycling in 0.8mm wide constant cracks. A similar behavior was observed when anchors were subjected to load cycling to a constant maximum load or to a stepwise increasing maximum load. They also conclude that approval tests conducted using stepwise load cycling to a maximum load of $0.5N_{um}$ (ACI355.2, 2007) do not provide meaningful results. They instead suggest stepwise loading to failure of the anchor.

**Mahrenholtz, (2009a)**

Mahrenholtz, (2009a) investigated the performance of different anchor types installed in 0.5 or 0.8mm cracks and subjected to either cyclic tensile or cyclic shear loads. The anchor types included wedge-type expansion, sleeve-type expansion, undercut, and epoxy anchors. Mahrenholtz investigated the effect of different stepwise increasing loading protocols including the FEMA 461 load protocol and the load protocol developed by Wood et al. (2009). The residual load capacity after load cycling was observed to decrease by about 20% when the crack width was increased from 0.5 to 0.8mm. The author concludes that bonded anchors exhibit good resistance to tensile load cycling. In addition, it was observed that the load cycling protocol did not significantly affect the mean load-displacement curves.
1.4.2 Constant Load with Crack Width Cycled

Hoehler and Eligehausen, (2008a)

Hoehler and Eligehausen (2008a) describe the testing and observed behavior of cast-in-place and post-installed anchors in cracked concrete where the cracks are repeatedly opened and closed while the load is held constant. They tested headed studs and four types of post-installed anchors including undercut, sleeve, expansion, and concrete screw anchors under simulated seismic crack cycling conditions. The considered ten cycles of crack opening to 0.8 mm (0.03 in.) and crack closing to 0.0 mm. They determined that the behavior of anchors under simulated seismic crack cycling depends on the anchor failure mode. Furthermore, behavior can be categorized based on the amount of displacement during crack cycling relative to the displacement at ultimate load in a corresponding static pullout test in an open crack. They concluded that anchors failing by concrete cone breakout can undergo displacements during crack cycling greater than those recorded at ultimate load in corresponding reference tests without significant reduction of the residual strength. They also concluded that the load-displacement curve for mechanical anchors failing by pull-through in crack cycling tests is bounded by the monotonic envelope from corresponding reference tests.

Mahrenholtz, (2009b)

Mahrenholtz, 2009b investigated the performance of cast-in-place and post-installed anchors installed in 0.5 or 0.8mm cracks and subjected to cyclic cracks under a reference crack cycling protocol developed by Wood et al. (2009). The crack protocol is a decreasing number of crack cycles at stepwise increasing crack widths with 32 crack cycles. The cracks were cycled open and alternating compressed closed. A constant tension load was applied during crack cycling equal to 40% of the monotonic reference test ultimate load in static
cracks. Approximately 100 tests were performed. The anchor types studied included headed bolts, expansion, sleeve, screw, undercut, and epoxy anchors. It was concluded that the expansion anchor had the largest residual displacement and the undercut and screw anchors had the least residual displacement. It was concluded that the residual displacements in 0.5 and 0.8mm crack widths were similar; however, the residual load capacity decreased between 13% and 22%. The residual tension load capacities were sometimes higher (+28%) and sometimes lower (-29%) than comparable reference test in static cracks. It was concluded that an important factor for the rate of displacement during crack cycling was the capability of reversed displacement during crack closing.

1.4.3 Load Rate Effects

Rodriguez et al. (2001)

The rate of loading will also affect the behavior of an anchor. Rodriguez et al. (2001) investigated the dynamic tensile behavior of post-installed anchors in concrete using a load rise time of 0.1 seconds, where load rise time is defined as the duration between the time of zero load to a subsequent maximum load (analogous to ¼ of a sine cycle). Anchor types investigated included; cast-in-place headed, expansion, sleeve, grouted, and undercut anchors. The anchors were embedded in both uncracked and cracked concrete (static, 0.4 mm width). All specimens used a 4 inch embedment depth in order to study concrete breakout. They determined that the tensile capacity changed by the following amount under dynamic loading in uncracked concrete for the various anchor types: undercut anchors 19 to 24% increase, grouted anchors 38% increase, sleeve anchors 3 to 24% increase, and expansion anchors 3% increase. They determined that the tensile capacity changed by the following amount under dynamic loading in cracked concrete having a width of 0.4 mm for the various anchor types:
undercut anchors 11 to 15% increase, grouted anchors 37% decrease, sleeve anchors 0 to 12% increase, and expansion anchors 6% decrease. It is of interest to note that in some cases, the ultimate displacement increased under dynamic loading and in some cases it decreased.

**Hoehler et al. (2011)**

Hoehler et al. (2011) studied the behavior of post-installed anchors under seismic relevant load rates. Anchor types included adhesive, torque controlled adhesive, sleeve, and expansion anchors. One of the unique features of this study is that it separates load rate behavior by failure mode type. Tests results for different crack widths are overlaid making it more difficult to generalize the cracked concrete case. Importantly, the authors find that anchor tension strength generally increases when the anchor is loaded repeatedly under earthquake relevant loading rates. For example, for concrete cone failure, the tensile strength increase when the anchor is loaded dynamically at a rise time of 0.1 seconds ranges from approximately 5 to 35% with an average of about 15% (Figure 1.9). They conclude that there is no clear trend for pull-through failure except that the strength was not decreased more than the normal scatter of typical anchor test results; however, no clear strength increase trend was observed across all data sets either. The results from Hoehler et al. (2011) for concrete cone and pull-through failure are particularly relevant for comparison to the shake table study results of this dissertation and are reproduced in Figure 1.9 and Figure 1.10, respectively.
Figure 1.9 Load rate effect on concrete cone failure (Hoehler et. al 2011)

Figure 1.10 Load rate effect on concrete pull-through failure (Hoehler et. al 2011)
1.4.4 Anchor Behavior Numerical Modeling

Numerical models of anchor behavior may be generally classified into two categories as follows: *micro* models and *macro* models. Micro models seek to simulate the physical mechanisms that occur during loading of an anchor, (e.g. concrete bearing, local concrete crushing, concrete cracking, steel yielding, friction between steel-to-steel elements, and friction between steel-to-concrete, etc...). The micro behavior of concrete and other brittle materials has been studied using finite element models. These finite element models have used the following approaches: (1) continuum damage mechanics (Hayhurst, 1972), (2) smeared crack models using fixed and rotating cracks (Jirasek and Bazant 1994), and (3) microplane models (Bazant and Oh, 1983; Ozbolt and Eligehausen, 1995; Ozbolt et al., 2001 and 2006). On the other hand, *macro* behavior of anchors has been studied by Dowell (2009) using a lumped plasticity approach to anchor modeling within the context of studying amplification factors for nonstructural components including the inelastic behavior of the anchors. The macro model for anchors is analogous to the approach commonly used for modeling plastic hinges in nonlinear analysis of buildings whereby the complex physical mechanisms that occur in plastic hinges (e.g. concrete crushing, reinforcement bar yielding, reinforcement bar buckling, etc…) are simplified by using a macro model of moment-rotation behavior for a reinforced concrete member.

1.5 Scope of Dissertation

Including the overview and introductory material of this chapter, this dissertation is arranged into ten chapters as follows. Chapter 1 provides introductory and background information while clearly identifying the objective of this work. Chapter 2 summarizes the design, construction, and assembly of the experimental equipment as well as provides
background for the selected testing parameters. Chapter 3 describes the monotonic tension behavior of anchors in free-field unconfined uncracked concrete and when located in static cracks. The anchor types are the same as those used in the shake table tests, namely epoxy, expansion, drop-in, and undercut anchors. Chapter 4 presents an overview of the shake table testing program of a model NCS anchored in cyclically cracked concrete including the test matrix, test specimens, test procedures, input floor accelerations, input crack width histories, and data post-processing techniques. Chapter 5 presents the system dynamic characterization results in terms of natural period and damping for both uncracked and cracked concrete conditions. Chapter 6 presents the results of shake table failure tests on different post-installed anchor types (epoxy, expansion, drop-in, and undercut anchors). Chapter 7 compares the anchor behavior and system performance between different failure tests. Chapter 8 describes trends in the correlation between anchor load and crack width. Chapter 9 describes the development of a numerical model for nonlinear anchor behavior and comparison of simulation and experimental results. Impacts on design practice and recommendations for future research are presented in Chapter 10. Finally, three Appendices are included (A, B, & C) to provide detailed design drawings of the model NCSs and CCILR as well as summary test data sheets.
Chapter 2 Experimental Equipment: Design and Construction

2.1 Background Design Parameters

In support of the experimentation, laboratory equipment in two main areas was designed and constructed. The first area was an experimental frame capable of producing dynamically cycled cracks, for this a Cyclic Crack Inertial Loading Rig (CCILR) was developed. To transfer inertial loads to the anchors two model NCSs were developed, a tension dominated system, Weighted Anchor Loading Laboratory Equipment (WALLE) and a shear dominated system (Shear Sled). The following sections describe the background parameters that were considered in the design of the CCILR and the model NCSs as well as the design, construction and assembly.

2.2 Variables of Interest to the CCILR

The purpose of the CCILR is to simulate dynamic crack cycling in concrete while being able to impose inertial loads to an anchored component, therefore, the main variables of interest are the maximum crack width ($w_{\text{max}}$), the period of crack opening and closing, herein referred to as the predominant period of cracking ($T_{p,cr}$) and the effective duration of cracking ($t_{d,cr}$). These variables and their ranges are explored in greater detail in the following subsections.

2.2.1 Maximum crack width $w_{\text{max}}$

As structures deform under seismic action, the structural members experience internal tensile strains due to shear, bending, and axial modes of deformation that cause the concrete to
A key parameter describing the crack is the crack width \((w)\). Crack width, as characterized from bending- or axial-induced strain, is well known to be sensitive to a variety of parameters including reinforcement ratio, depth of cover, spacing of reinforcement, reinforcing bar diameter, reinforcement bond characteristics, concrete tensile strength, depth to neutral axis, and strain in the extreme tension fiber of the beam (e.g. Broms, 1965; Broms and Lutz, 1965; Nawy, 1968; Gergely and Lutz, 1968; Rehm and Martin, 1968; Orenstein and Nawy, 1970; Martin and Schwarzkopf, 1980; Bazant and Oh, 1983; Oh and Kang, 1987; Frosch, 1999; Marzouk et al., 2010). Due to the variability of these parameters in practice as well as the uncertain nature of earthquake motions, the maximum width of the cracks \(w_{max}\) that develop in a concrete structure may vary greatly. Furthermore, the characteristics of shear cracks will differ from those caused by tension or bending. If one focuses on post-installed anchors, however, it is recognized that they are commonly located in floor slabs and beams. Moreover, for anchored NCSs, code provisions require that anchors be installed outside of plastic hinge zones. In this context, the manifestation of cracking is largely due to beam or slab flexural deformations induced by lateral earthquake motions.

There is debate regarding the maximum crack width that may occur in a concrete structure under seismic loading. Outside of the plastic hinge zone, the maximum flexural crack width occurs at insipient yielding of the reinforcement (Hoehler, 2006). In this context one may relate \(w_{max}\) to beam or slab yield curvature. Wood et al. (2009) for example, finds \(w_{max}\), as associated with beam yielding, to range from 0.4 to 0.6 mm (0.016 to 0.024 in.), while Hoehler (2006) found this range to be from 0.4 to 1.0 mm (0.016 to 0.04 in.). These and other experimental and analytical investigations have suggested that properly designed and detailed members will experience maximum flexural crack widths at incipient yielding of the reinforcement on the order of 0.4 to 1.0 mm (0.016 to 0.04 in.). Based on these observations,
the target maximum crack width for the experimental system developed herein was selected as 1.0 mm (0.04 in.).

2.2.2 Crack cycling predominant period $T_{p,cr}$

Flexural crack width time histories in structural members, which often serve as the anchorage base material, are dependent on the dynamic displacement response of the primary structure. Thus, important characteristics used to describe crack cycling are the predominant period of cracking $T_{p,cr}$ and the relationship between $T_{p,cr}$ and the predominant period of the structure, $T_n$. The crack cycling predominant period is extracted from the member normalized curvature time history $\lambda_{cr}(t)$ by indentifying the periods of the peak of the magnitude of the Fourier spectrum $|\text{FFT}[\lambda_{cr}(t)]|$, where $\lambda_{cr}$ is defined as $\lambda_{cr}(t) = \phi(t)/\phi_y$ which is approximately equal to $w(t)/w_{max}$, where $\phi(t)$ is the curvature time history, $\phi_y$ is the yield curvature, $w(t)$ is the crack width history, and $w_{max}$ is the maximum crack width.

The $T_{p,cr}$ results from five building models subjected to all 21 selected earthquake motions are presented in Figure 2.1. Table 2.1 and Figure 2.1 indicate that $T_{p,cr}$ typically corresponds to the first modal period of vibration of the building ($T_1$), but may shift by up to 10% due to nonlinear behavior of the building. For taller structures where higher mode effects contribute, $T_{p,cr}$ can also correspond to the second modal period of vibration ($T_2$) of the building (Figure 2.1; 12 and 20 story buildings). As a consequence of this multi-mode contribution to the vibration response, $T_{p,cr}$ may vary with elevation in the building, particularly for taller buildings.
Table 2.1 Average predominant period of cracking ($T_{p,cr}$) and comparison with elastic first modal period of building models based on simulation results of Wood et al. (2009)

<table>
<thead>
<tr>
<th>Parameter of Interest</th>
<th>2 story</th>
<th>4 story</th>
<th>8 story</th>
<th>12 story</th>
<th>20 story</th>
</tr>
</thead>
<tbody>
<tr>
<td>$T_{p,cr}$ (sec)</td>
<td>0.26</td>
<td>0.49</td>
<td>0.97</td>
<td>1.43</td>
<td>2.03</td>
</tr>
<tr>
<td>$T_1$ (sec)</td>
<td>0.24</td>
<td>0.47</td>
<td>0.89</td>
<td>1.33</td>
<td>2.07</td>
</tr>
<tr>
<td>$T_{p,cr}/T_1$</td>
<td>1.08</td>
<td>1.05</td>
<td>1.09</td>
<td>1.07</td>
<td>0.98</td>
</tr>
</tbody>
</table>

Figure 2.1 Predominant Period of Cracking ($T_{p,cr}$) versus maximum normalized crack width ($\lambda_{cr,max}$) normalized by maximum $\lambda_{cr}$ of five building models of height 2, 4, 8, 12 and 20 stories subjected to 21 earthquake motions.
2.2.3 Duration of effective strong crack cycling $t_{d,cr}$

Anchor performance is affected by the number of crack cycles and effective duration of crack cycling. One way to characterize the effective duration of crack cycling is to consider the time over which the significant amplitude crack widths occur. One may analogize the problem to methods for characterizing the duration of significant strong earthquake shaking (Trifunac and Brady, 1975; Bommer and Martinez-Periera, 1999). Adopting an energy-based concept in this context, the duration of effective strong crack cycling is defined as the duration between the time intervals associated with the 5% and 95% cumulative Arias intensity (Arias, 1970), where the Arias intensity is reformulated using the normalized crack width time history ($\lambda_{cr}$) as follows:

$$I_{\lambda_{cr}} = \int_{0}^{t_{end}} [\lambda_{cr}(t)]^2 dt$$  

where $I_{\lambda_{cr}}$ = Arias intensity of the normalized cracking time history, $t_{end}$ = total duration of the record and $\lambda_{cr}(t)$ = normalized crack history. Table 2.2 summarizes the effective duration of strong cracking calculated using the analyses results from Wood et al. (2009). These data indicate the range of average $t_{d,cr}$ is between 24.1 and 6.7 seconds, and generally decreases with increasing building height. Of the five buildings, the largest mean plus one standard deviation ($\mu + \sigma$) duration is 30.4 seconds, therefore the performance goal for the CCILR is selected to capture this threshold and thus reproduce crack time histories with durations of strong cracking in excess of 30 seconds.
Table 2.2 Effective duration of cracking \( (t_{d,cr}) \)

<table>
<thead>
<tr>
<th>Parameter of Interest</th>
<th>2 story</th>
<th>4 story</th>
<th>8 story</th>
<th>12 story</th>
<th>20 story</th>
</tr>
</thead>
<tbody>
<tr>
<td>( (t_{d,cr}) ) Average (( \mu ))</td>
<td>24.1</td>
<td>8.9</td>
<td>8.9</td>
<td>6.7</td>
<td>11.3</td>
</tr>
<tr>
<td>( (t_{d,cr}) ) Std. Dev. (( \sigma ))</td>
<td>6.3</td>
<td>8.2</td>
<td>0.9</td>
<td>0.5</td>
<td>3.2</td>
</tr>
<tr>
<td>( (t_{d,cr}) ) Minimum</td>
<td>18.3</td>
<td>4.3</td>
<td>8.2</td>
<td>6.3</td>
<td>7.0</td>
</tr>
<tr>
<td>( (t_{d,cr}) ) Maximum</td>
<td>29.7</td>
<td>22.3</td>
<td>11.5</td>
<td>8.6</td>
<td>22.8</td>
</tr>
<tr>
<td>( (t_{d,cr}) \mu + \sigma )</td>
<td>30.4</td>
<td>17.1</td>
<td>9.8</td>
<td>7.2</td>
<td>14.5</td>
</tr>
</tbody>
</table>

### 2.3 Characterization and Parameterization of NCSs

A significant amount of background work regarding classification of NCSs has been performed, leading up to the design of several model NCSs for laboratory investigation, including a tension dominated system (WALLE) Weighted Anchor Loading Laboratory Equipment and a shear dominated system (Shear Sled). The background work included a detailed survey and categorization of NCSs in six university buildings, one hospital, and one office building. The focus of the survey was on floor and wall-mounted mechanical and electrical equipment in buildings. The study included 1093 NCSs, of these, 166 NCSs had a detailed survey of dimensions, weight and anchorage, and 19 NCSs tested using impact hammer modal tests to determine natural frequency and damping (Watkins et al., 2009).

Findings from the study indicate that the majority of NCSs are mounted to concrete by means of post-installed anchors, typically single anchors at each of the four corners. Groups of closely spaced anchors were rarely encountered. Typically \( \frac{1}{2}'' \) diameter expansion anchors were used, spaced far apart such that group effects do not influence the behavior. The majority of the NCSs surveyed (84%) weighed less than 2000 pounds with 37% considered “light,” i.e., weigh less than 400 pounds. Over half of the NCSs (53%) were considered large, with at least one plan dimension exceeding 48 inches. The aspect ratio, defined as height to width or depth in plan, exceeded 3 to 1 in any direction for less than 20% of the surveyed equipment, in other
words, most of the equipment is considered to have a relatively squat aspect ratio. The low aspect ratio of most NCSs combined with the observation that the elevation of the center of gravity of the equipment typically ranges from 1/2 to 2/3 of the height of the equipment implies that the majority of floor-mounted equipment are dominated by shear actions during a seismic event. Damping for the equipment that was hammer tested was on average 2% of critical in the low level hammer tests.

An online database of NCSs with Special Seismic Certification preapproval in California hospital buildings (OSHPD, 2010) as well as field investigation of real ME systems in buildings (Watkins et al., 2009) was used to determine the range of natural periods of typical ME-type NCSs. A histogram of the distribution of natural frequency is shown in Figure 2.2. Typically, the frequency was measured in the sided-to-side direction as well as the front-to-back direction of the NCS, therefore, each measurement is included in the figure as a separate occurrence. The average period of NCSs in the database is 0.11 seconds (9.1 Hz). Using a range of +/- one standard deviation from the average, the period range is 0.07 seconds to 0.25 seconds (14 Hz to 4 Hz).
2.4 Model NCS Design

Adopting the observations from the survey results and database, two reusable, configurable, model NCSs were designed: 1) a squat aspect ratio NCS designed to load the anchors predominantly in shear and 2) a tall aspect ratio NCS designed to load the anchors predominantly in tension. Tension dominated anchorage is much more affected by the presence cyclic cracking, therefore to illustrate the complete system methodology, the tension dominated model NCS is discussed in further detail.

2.4.1 Weighted Anchor Loading Laboratory Equipment (WALLE) Design

The tension dominated NCS, shown in Figure 2.3, is herein referred to as the Weighted Anchor Loading Laboratory Equipment (WALLE). Constructed of steel tube and angle sections, WALLE is designed to transfer loads to 4-single anchor Shear and Axial
Measurement Units (SAMUs) placed at its base (Hoehler et al., 2011). Varying the mass of WALLE allows one to tune its frequency as well as vary the inertial loading transferred to the anchors. Two WALLE configurations were used during testing, *stiff* and *flexible*, with natural periods of vibration of 0.10 and 0.25 seconds, and weights of 390 kg (860 lb) and 1160 kg (2550 lb) respectively. This period bound encompasses approximately 50% of the cases shown in Figure 2.2. A full set of fabrication drawings of the WALLE assembly is available in Appendix A.

**Figure 2.3 Weighted Anchor Laboratory Loading Equipment (WALLE) models a) stiff and b) flexible configuration and (c) photograph of flexible configuration**

The WALLE was designed to accommodate various anchor spacing configurations to investigate both single anchor and group behavior. In the direction perpendicular to the direction of motion, multiple bolt hole locations in the WALLE base tub (Appendix A) allow the anchors to be spaced at 8”, 13”, 24”, and 36” respectively. In the test series described herein the anchors were always spaced at 24” in the direction perpendicular to the direction of
motion and 13” in the direction of motion. The failure cone is defined by ACI 318 (2008) as a cone with diameter at the surface of the concrete equal to $3h_{ef}$ where $h_{ef}$ is the effective embedment depth of the anchor. This geometry corresponds to a cone originating at the head of the anchor sloping at an angle of approximately 35 degrees with respect to the plane of the concrete surface. Figure 2.4 describes the influence of the failure cones for the anchors tested. This figure demonstrates that the stress cones were sufficiently far apart during testing to support the assumption of single anchors in free-field unconfined concrete.

2.4.2 Shear Sled Design

A second model NCS was designed and constructed to load anchors predominantly in shear and is herein called the Shear Sled. The shear sled was not used during this phase of
testing; however, its basic design and construction were performed in this stage of the project to compliment the overall research project, therefore it is discussed herein. The Shear Sled was designed to accommodate multiple SAMU positions as well as adjustable height of the SAMUSs so that the base of the shear sled could be elevated, providing high confining pressure around anchors, or flush with the concrete providing low confining pressure at the anchor locations. The shear sled loaded with approximately 5000 lbs of weight plates and mounted on the shake table is shown in Figure 2.5. A full set of fabrication drawings of the shear sled assembly is available in Appendix A.

![Figure 2.5 Shear sled with weight plates mounted on the shake table](image)

2.4.3 SAMU Overview

Four Shear and Axial Measurement Units (SAMUs) were developed by Hoehler et al. (2011) to transfer inertial loads from the WALLE into the anchors. Hoehler et al. (2011) fully
describe the construction, calibration and experimental validation of a test apparatus to measure axial and shear forces in anchors. Figure 2.6 shows a conceptual sketch of the SAMU design. The device consists of four parts: fixture; pins; collar; and load washer. The fixture is composed of a 6”x6”x1” piece of steel angle cut to 6” in length. Anchor axial and shear forces are measured independently. Axial forces are measured using a commercially available through-hole load cell (load washer). Transverse movement of the fixture relative to the anchor axis causes shearing in the anchor, which in turn generates axial strains in the four pins. Strain gages are embedded in each of the four pins. Key dimensions of the SAMU fixture are given in Figure 2.7. Photographs of the components as well as a fully assembled SAMU are shown in Figure 2.8.

![Figure 2.6 Conceptual sketch of SAMU design: (a) elevation view; (b) plan view (from Hoehler et al. 2011)](image-url)
Figure 2.7 SAMU fixture dimensions: (a) elevation view; (b) plan view, units in inches
(from Hoehler et al. 2011)

Figure 2.8 Photograph of SAMU and components: (a) fixture; (b) pin and collar assembly; (c) assembled with anchor and tension load ring (from Hoehler et al. 2011)
2.5 Cyclic Crack Inertial Loading Rig (CCILR) Design, Construction, and Assembly

A key component to this effort is a test fixture, which will generate simultaneous inertial loading of the anchor and cyclic cracking. The CCILR load system is designed to simulate a portion of a reinforced concrete beam or floor slab in a building subjected to a strong earthquake motion. The test fixture and protocol together allow for investigation of combined component-anchor system behavior and component inertial response as modified by dynamically opening and closing cracks. In this section we present details of the design, construction, and assembly of such a test fixture. First however, the shake table is discussed, which the CCILR is designed to mount to.

2.5.1 USCD Powell South Laboratory Shake Table

The Powell lab facility has a uniaxial shake table that provides shaking in the North-South direction. The platen plan dimensions are 16 ft x 10 ft with a 10” center to center grid of 5/8” diameter mounting holes. The table is capable of reproducing earthquake input motions via an On-Line Iteration (OLI) or Adaptive Inverse Control (AIC) tuning procedure. AIC was used for the input motion tuning in this research. Figure 2.9 describes the shake table dimensions and relevant performance characteristics. The CCILR was designed to adapt to this table.
2.5.2 CCILR-WALLE System

The CCILR is mounted on a shake table, which provides the floor input motion to dynamically excite the model NCS (WALLE), this in turn loads the anchors in shear and tension. The primary purpose of the CCILR is to generate the cyclic crack time histories. The cyclic cracks are imposed onto a reinforced concrete slab. A conceptual design sketch of the CCILR-WALLE system mounted on the shake table is shown in Figure 2.10. A photograph of the as-built CCILR-WALLE system mounted on the shake table is shown in Figure 2.11.

In totality, the system consists of a reinforced concrete member and a frame coupled with a pair of servo-hydraulic actuators used to dynamically open and close cracks in the slab. To create seismic crack histories, two 734 kN (165 kip) servo-hydraulic actuators are attached to a concrete slab of dimensions 2438 x 1067 x 254 mm (90 x 42 x 10 in.) by means of four...
high strength reinforcing bars. The high strength reinforcing bars are bolted to fixed and sliding end beams. As the hydraulic actuators push the beams apart, the high strength bars are mobilized and cause tension in the slabs, in turn causing cracks to form at crack inducers located to coincide with the stabilized crack spacing of the slab. As the actuator load reverses and pulls the beams towards each other, the beams bear on the slab, placing it in compression, which actively closes the cracks. The CCILR is designed to mount on the Powell South Laboratory shake table, which provides the floor input motion to dynamically excite the NCS. This motion in turn loads the anchors in shear and tension. CCILR is configured to allow mounting of anchored NCSs on top of the slab, however it can be modified for suspended NCSs. The CCILR steel frame weighs approximately 5.6 kips. The concrete slab weighs 3.5 kips. The hydraulic actuators weigh approximately 2.0 kips each (4.0 kips total). The flexible WALLE as shown weighs 2.55 kips. The total weight of the combined CCILR-WALLE system as shown is approximately 15.6 kips.
Figure 2.10 Schematic of the Cyclic Crack Inertial Load Rig (CCILR) with the Model NCS attached, Weighted Anchor Laboratory Loading Equipment (WALLE) (assembly mounted on shake table).

Figure 2.11 Photograph of the Cyclic Crack Inertial Load Rig (CCILR) with the Model NCS attached, Weighted Anchor Laboratory Loading Equipment (WALLE).
To provide a clear understanding of the CCILR parts, the CCILR base frame without slab and actuators is shown in Figure 2.12. Figure 2.13 depicts the sliding end beam and Teflon® sliding pads, which reduce friction between the sliding end beam and the base frame. The rubber pads shown were inserted at each end of the concrete slab between the concrete slab and the CCILR beams to promote even bearing of the slab against the CCILR frame. A full set of fabrication drawings of the CCILR assembly is available in Appendix B.

Figure 2.12 Photograph of the Cyclic Crack Inertial Load Rig (CCILR) bare frame (shown without actuators or concrete slab for clarity)
2.6 Assembly of CCILR-WALLE System on the Shake Table

Testing of the entire CCILR-WALLE system on the shake table required additional design considerations for safety as well as mounting the CCILR to the shake table. Figure 2.14 describes the construction assembly sequence for CCILR. After CCILR is bolted to the shake table, the concrete slab is lifted into place. Then the sliding end beam is assembled, then the hydraulic actuators are installed. Figure 2.15(a) shows a conceptual design, which includes the following parts: 1) safety catch frame, 2) safety system and hoist, 3) WALLE, 4) CCILR, 5) shake table, 6) reaction wall and floor. Figure 2.15(b) shows a photograph of the as-built system. The reaction frame and safety system was designed to support the entire weight of WALLE accelerating at 4g’s laterally in the case of a complete failure of the anchorage. The hoist and trolley were designed to support placement of WALLE to different positions on the slab as well as removal of WALLE from the CCILR to allow for changing out of concrete slab
specimens. The CCILR was mounted to the shake table by means of 14 – 5/8” diameter grade 8 bolts with ¼” thick plate washers on top of the CCILR longitudinal beams and a single nut on the underside of the mild steel table platen. The reaction wall and floor served as fixed reference points for attachment of displacement instrumentation.
Figure 2.14 CCILR assembly sequence

(a) Concrete slab lifted into place

(b) Sliding end beam assembled

(c) Attach hydraulic actuators to CCILR
Figure 2.15 CCILR and WALLE assembly on the shake table (a) conceptual design and (b) photograph of as-built system

2.7 Acknowledgement of Publications

Parts of this chapter were published in the following journal papers in which the author of this report was either the primary or a co-author.


Chapter 3 Monotonic Tension Behavior of Anchors

3.1 Selection of Anchors

A total of four anchor types were tested in order to achieve a differing failure modes and tension load-displacement behavior. The anchor embedment was set such that all anchors had approximately the same ultimate strength in uncracked concrete. Concrete breakout failure was investigated using epoxy, drop-in, and undercut anchors. Pull-through failure was investigated using expansion anchors (Figure 3.1). The anchors were all commonly available off-the-shelf unmodified anchors except for the undercut anchor which had a modified sleeve to allow for a shallower than standard embedment depth. Additionally in an effort to distinguish between the performance of anchors with seismic approvals and anchors without seismic approvals, the drop-in anchors were selected as an anchor that does not have a seismic approval (ACI 355.2, 2007).
3.2 Test Program

To characterize tension load-displacement behavior, monotonic tension tests were conducted on single anchors in free-field unconfined normal weight, normal strength, uncracked and cracked concrete. The tests were conducted at the University of California, San Diego (UCSD) Powell laboratory. Anchors were loaded monotonically in tension to failure in normal strength uncracked and cracked concrete with static crack width equal to 0.8 mm (0.03 inches). Several tests were repeated in cracks with a static crack width equal to 0.5 mm (0.02 inches). A total of 29 tension tests were performed, 14 tests in uncracked concrete and 15 tests in cracked concrete. Hoehler (2009) conducted a companion test program on anchors in uncracked concrete at San Diego State University (SDSU) for the same project. Two of the
anchor types, epoxy and expansion anchors, were common to both test programs and results from the two are compared herein. This chapter summarizes the load-displacement behavior as characterized by the backbone curves, failure modes, and initial, ultimate and post peak loads, displacements, and stiffnesses of the various anchors tested.

Table 3.1 Tension test program

<table>
<thead>
<tr>
<th>Test Series</th>
<th>ID</th>
<th>Type / (Failure)</th>
<th>Diameter &amp; Embedment</th>
<th>Crack Width Inch [mm]</th>
<th>No. Tests</th>
</tr>
</thead>
<tbody>
<tr>
<td>T_UC_12Epoxy</td>
<td>N4</td>
<td>Epoxy (C)</td>
<td>$d = 0.5''$</td>
<td>uncracked</td>
<td>3</td>
</tr>
<tr>
<td>T_CC08_12Epoxy</td>
<td>N4</td>
<td>Epoxy (C)</td>
<td>$h_{ef} = 2.4''$</td>
<td>0.03'' [0.8]</td>
<td>3</td>
</tr>
<tr>
<td>T_UC_12Expansion</td>
<td>N5</td>
<td>Expansion (Pt)</td>
<td>$d = 0.5''$</td>
<td>uncracked</td>
<td>4</td>
</tr>
<tr>
<td>T_CC05_12 Expansion</td>
<td>N5</td>
<td>Expansion (Pt)</td>
<td>$h_{ef} = 3.75''$</td>
<td>0.02'' [0.5]</td>
<td>3</td>
</tr>
<tr>
<td>T_CC08_12 Expansion</td>
<td>N5</td>
<td>Expansion (Pt)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>T_UC_12Drop-In</td>
<td>N7</td>
<td>Drop-in (C)</td>
<td>$d = 0.5''$</td>
<td>uncracked</td>
<td>4</td>
</tr>
<tr>
<td>T_CC08_12Drop-In</td>
<td>N7</td>
<td>Drop-in (C)</td>
<td>$h_{ef} = 2.0''$</td>
<td>0.03'' [0.8]</td>
<td>3</td>
</tr>
<tr>
<td>T_UC_M10Undercut</td>
<td>N9</td>
<td>Undercut (C)</td>
<td>$d = 0.394''$</td>
<td>uncracked</td>
<td>3</td>
</tr>
<tr>
<td>T_CC08_M10Undercut</td>
<td>N9</td>
<td>Undercut (C)</td>
<td>$h_{ef} = 2.4''$</td>
<td>0.03'' [0.8]</td>
<td>3</td>
</tr>
</tbody>
</table>

All tests performed in $f_c = 4000$ psi nominal strength concrete
(1) N4 and N5 anchors also tested by Hoehler and Dowell 2009
Failure mechanisms: (C) Concrete breakout, (Pt) Pull-through
as defined in ACI 355.2, 2007.

Table 3.1 summarizes the testing program. A total of 29 tests were performed. Tension testing using monotonic loading to failure was performed in normal weight, 4000 psi compressive strength concrete slabs that were specifically designed for performing tests in cracked concrete. The anchor spacing, edge distance, and clear distance from the anchor to the loading frame were selected to allow for uninfluenced single anchor failure (i.e. free-field, unconfined tension tests). All anchors were tested in both uncracked concrete and concrete with static crack width of 0.03 inch [0.8 mm]. In addition, the expansion anchor was also tested in 0.02 inch [0.5 mm] cracks to study the sensitivity of the pull-through failure mode to crack width.
3.2.1 Adhesive anchor, concrete breakout failure

The anchors used to study concrete breakout failure were constructed of 1/2” diameter ASTM A193 (2009) B7 thread rod ($f_u = 125$ ksi) embedded in epoxy mortar (Figure 3.2). A bonded length of 2.4” was used. The embedment depth was selected in order to achieve a concrete breakout failure (i.e. steel and bond failure did not control). There were 2.75” of free length above the surface of the concrete to accommodate the loading fixture and load washer.

Figure 3.2 Adhesive anchor, concrete breakout failure
3.2.2 Torque-controlled expansion anchor, pull-through failure

The anchors used to study pull-through failure were torque-controlled expansion anchors (wedge-type), herein termed expansion anchors. They consist of a specially formed steel bolt with a conical end, and expansion element, a washer and a hexagonal nut (Figure 3.3). The tested anchors were size 1/2"x7" (diameter x length) and made of carbon steel ($f_u = 106$ ksi) zinc plated to 5 microns. Embedment depth of 3.75 inches was used. The embedment depth was selected in order to achieve a pull-through failure. There were 3.25 inches of free length above the surface of the concrete to accommodate the loading fixture and load washer.

![Figure 3.3 Torque-controlled expansion anchor, pull-through failure](image-url)
3.2.3 Drop-in anchor, concrete breakout failure

The anchors used to study concrete breakout failure were drop-in anchors. They consist of a steel shell with expansion mechanism and a 1/2 inch diameter ASTM A193 (2009) B7 thread rod ($f_u = 125$ ksi) that screws into the steel shell (Figure 3.4). The tested anchors have an outer shell diameter of 5/8 inches, which accommodates a 1/2 inch diameter threaded rod. An effective embedment depth of 2 inches was used. The embedment depth was selected in order to achieve a concrete breakout failure. The top of the shell was set flush with the surface of the concrete. The threaded rod was screwed into the shell as far as possible. There were approximately 2 inches of free length of the thread rod above the surface of the concrete to accommodate the loading fixture and load washer.

![Figure 3.4 Drop-in anchor, concrete breakout failure](image)

Figure 3.4 Drop-in anchor, concrete breakout failure
3.2.4 Undercut anchor, concrete breakout failure

The anchors used to study concrete breakout failure with load resisted at the head of the anchor were undercut anchors modified to provide a reduced embedment depth. They consist of a specially formed steel bolt with a conical end, and undercutting element, a washer and a hexagonal nut (Figure 3.5). The steel is Grade 8.8 ($f_u = 116$ ksi) galvanized to minimum 5 microns. The tested anchors were galvanized steel, size M10 [0.394 inch] which is the diameter of the threaded rod. The effective embedment depth was 2.4 inches. The standard M10 anchor has an embedment depth of 3.937 inches. The body of the tested anchors was milled down in length to 2.4 inches and the setting “key” was specially milled into the body. There were 3.25 inches of free rod length above the surface of the concrete to accommodate the loading fixture and load washer.
3.3 Concrete Slabs

3.3.1 Details and Production

Two specialty concrete slabs (denoted slab 3 and slab 4) were produced to serve as the anchorage material for the monotonic reference tests. The slabs were formed and placed at the University of California, San Diego (UCSD) and the concrete was produced by Vulcan Materials Company of San Diego.

The Vulcan City Mix Design 202502CD used for the slabs is given in Figure 3.6. It is noted that 10 gallons of water were “held back” at the batch plant and 10 gallons were added at the pour site (UCSD), so the actual mix had the same amount of water as shown in Figure 3.6. The resulting water cement ratio was 0.62. The measured slump was approximately 3 inches.
The slabs were 10 inches thick and were designed to produce a 13 inch crack spacing as shown in Figure 3.7 and Figure 3.8. The longitudinal reinforcement of the slabs consists of four 7/8 in. nominal diameter high strength reinforcing bars (DYWIDAG Threadbar®) with a specified ultimate stress $f_u$ of 160 ksi and specified yield stress $f_y$ of 130 ksi (Figure 3.8). Reinforcement was placed such that it did not intersect the anchor failure cone, therefore it did not influence the anchor behavior. Three No. 3 reinforcing hoops with a nominal yield stress of $f_y$ of 60 ksi were provided at the ends of the slab to provide confinement. Stainless steel sheet metal crack inducers (thickness = 0.11 in.) are placed transversely in the slab formwork to aid crack formation (Figure 3.9).
Figure 3.7 Photograph of slabs 3 and 4 formwork

Figure 3.8 Drawing of slabs 3 and 4

NOTES:
1) CONCRETE: 3000 PSI (min), 4000 PSI (max).
2) "VULCAN – 2029GCD CITY MIX DESIGN"
3) SLUMP 3"-4"
4) TIE REINFORCEMENT = ASTM A 615, GR 60

SLAB 3 & 4 - 13" Crack Spacing

Crack inducers
12 gauge stainless steel sheet metal

Debonding sleeves at desired crack locations

Lifting inserts

High strength reinforcing bars 0.787 inch dia. \( f_y = 160 \text{ksi} \)

3 - #3 ties

7/8" Diameter "DYWIDAG THREADBAR" Form Tie Bar

2 - 1/2" Triangular Molding Tack-nailed into Formwork (TYP)

Debonding Sleeve. PVC 1.25" Dia. Sch. 80 (OD=1.66", ID=1.255") sealed at ends & centered on crack inducer

Crack Inducer (12 GA Galvanized Sheet Metal)

First hoop 2" from end of slab

Lifting Insert (4 PLCS) in side of slab

SL4 SL4 SL4 SL4 SL4 SL4 SL4 SL4

SL4 SL4 SL4 SL4 SL4 SL4 SL4 SL4

SL4 SL4 SL4 SL4 SL4 SL4 SL4 SL4

SL4 SL4 SL4 SL4 SL4 SL4 SL4 SL4
Anchor test locations observed minimum edge distance and spacing requirements. Anchors for uncracked tests were located in the free space on the slab between cracks. Anchors for cracked tests were located in the cracks. This was accomplished by pre-drilling
the slabs with a series of pilot holes, ¼ inch diameter, 4 inches deep, spaced at 3” on center transversely along the cracks (Figure 3.10). Cracks were initiated in the slabs by applying tension to the four high-strength DYWIDAG bars. The appropriate diameter hole for the anchor under consideration was drilled using the pilot holes as a location guide. All drill holes were visually inspected using a borescope to ensure that the cracks were located in the drill holes.

### 3.3.2 Hardened Concrete Strength

The uniaxial unconfined compressive strength of the concrete was determined using standard 6”x12” cylinders cured indoors in open air and tested according to standard practice. The measured cylinder strengths are reported in Figure 3.11. The target strength of the slabs was 3000 to 4000 psi. Monotonic tension testing was conducted from 10/06/2009 to 01/15/2010. Slabs 3 and 4 had a measured compressive strength of approximately 4000 psi.

![Figure 3.11 Concrete compressive strength, slab 3 and 4](image-url)
3.4 Anchor Installation

All anchors were installed by first drilling a hole into the concrete slab to the required depth using a Hilti TE40-ARV rotary-hammer and carbide tipped drill bit with the required cutting diameter (refer Table 3.2 for required dimensions). A drilling stand was used to ensure that the hole was drilled perpendicular to the concrete surface (Figure 3.12). During and after drilling, a vacuum was used to remove excess concrete debris. After drilling, the hole was thoroughly cleaned using compressed air by inserting and withdrawing an air lance from the hole 3 times. This procedure was repeated 3 times. The holes for the bonded anchors were also brushed with a wire brush in between the 3x blowing by inserting and withdrawing the rotating brush 3 times, referred to as blow 3x, brush 3x, blow 3x, brush 3x, blow 3x. As noted previously, this cleaning was in excess of the manufacturer’s instructions and was performed to achieve optimal bond conditions. The drill hole depth was measured as the depth from the concrete surface to a point along the perimeter edge of the drill hole. Two points were measured on opposite side of the drill hole and the average value was recorded.
For the epoxy anchors, the threaded rods were cleaned using Acetone to remove any oil and dirt. The cleaned drill hole was then injected with the epoxy mortar from the bottom up using a “piston plug” assembly provided with the anchor system to help eliminate air pockets. The thread rods were then inserted into the mortar and excess mortar expressed from the drill hole was removed from around the surface of the hole. The anchor shaft was squared to the concrete surface and left untouched for the required curing time, which was 12 hours for epoxy anchors. After curing of the mortar the load fixture and instrumentation was attached.

For the investigated mechanical anchors, expansion, drop-in, and undercut anchors, the anchors were hammered into the cleaned drill hole to the required embedment depth using a metal hammer taking care not to damage the threads. The anchors were “set” according to the manufacturer’s instructions. The load fixture, load washer, spherical washer and nut were then placed on the anchor and the nut was tightened with a torque wrench with digital readout to the required installation torque. After at least 5 minutes and immediately prior to the test the pre-tension was removed completely by backing off the nut. Torque was then reapplied to
50% of the required installation value, applicable only for expansion and undercut anchors, to simulate relaxation over time.

The anchors were installed in accordance with the manufacturer’s recommendations with the following modifications / exceptions: 1) all drill holes (with the exception of epoxy) were cleaned using a sequence of 3x blowing. The hole preparation procedure for epoxy consisted of 3x blowing, 3x brushing, 3x blowing, 3x brushing, 3x blowing, to enhance bond to the hole, 2) the threaded rod for the epoxy anchors were cleaned with acetone to remove dirt and oil, 3) the expansion anchors were set ½” deeper than the recommended depth to ensure pull-through failure 4) The undercut anchors were modified to have a 2.4 inch embedment depth and 5) all installation torques (where relevant, expansion and undercut) were reduced to 50% \( T_{\text{inst}} \) according to the recommended procedure of ACI 355.2, 2007 immediately prior to testing to account for relaxation and loss of preload with time where \( T_{\text{inst}} \) is the manufacturers recommended installation torque. The expansion and undercut anchors were initially set to 100% \( T_{\text{inst}} \) then, prior to testing, the nuts were untightened completely, then 50% \( T_{\text{inst}} \) was applied. The nuts were assembled “finger-tight” for the other anchors (epoxy and drop-in). All anchors were located in the slab such that the minimum edge distance and spacing were observed in order to not influence the test results. A summary of the anchor installation parameters is provided in Table 3.2.
Table 3.2 Installation parameters

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Symbol</th>
<th>Units</th>
<th>Anchor ID</th>
<th>N4 Epoxy</th>
<th>N5 Expansion</th>
<th>N7 Drop-In</th>
<th>N9 Undercut</th>
</tr>
</thead>
<tbody>
<tr>
<td>Anchor ID</td>
<td></td>
<td></td>
<td></td>
<td>N4</td>
<td>N5</td>
<td>N7</td>
<td>N9</td>
</tr>
<tr>
<td>Anchor Type</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Thread rod diameter</td>
<td>(d)</td>
<td>[in]</td>
<td></td>
<td>0.5</td>
<td>0.5</td>
<td>0.5</td>
<td>0.394</td>
</tr>
<tr>
<td>Thread rod type</td>
<td></td>
<td></td>
<td></td>
<td>B7</td>
<td>n/a</td>
<td>B7</td>
<td>Gr 8.8</td>
</tr>
<tr>
<td>Effective depth</td>
<td>(h_{ef})</td>
<td>[in]</td>
<td></td>
<td>2.4</td>
<td>3.75</td>
<td>2.0</td>
<td>2.4</td>
</tr>
<tr>
<td>Drill hole depth</td>
<td>(d_{hole})</td>
<td>[in]</td>
<td></td>
<td>2.4</td>
<td>4.0</td>
<td>2.0</td>
<td>2.625</td>
</tr>
<tr>
<td>Drill bit diameter</td>
<td>(d_{cut})</td>
<td>[in]</td>
<td></td>
<td>0.5625</td>
<td>0.500</td>
<td>0.625</td>
<td>0.7874</td>
</tr>
<tr>
<td>Height above concrete</td>
<td>(h_{proj})</td>
<td>[in]</td>
<td></td>
<td>2.75</td>
<td>3.0 / 3.13(^{(1)})</td>
<td>0.00(^{(2)})</td>
<td>3.125</td>
</tr>
<tr>
<td>Cleaning</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>3x blow/brush</td>
<td>3x blow</td>
<td>3x blow</td>
</tr>
<tr>
<td>Installation torque</td>
<td>(T_{inst})</td>
<td>[ft-lb]</td>
<td></td>
<td>n/a</td>
<td>40</td>
<td>n/a</td>
<td>36</td>
</tr>
<tr>
<td>Reduced torque 50%</td>
<td>(T_{test})</td>
<td>[ft-lb]</td>
<td></td>
<td>n/a</td>
<td>20</td>
<td>n/a</td>
<td>18</td>
</tr>
<tr>
<td>Minimum slab thickness</td>
<td>(h_{min})</td>
<td>[in]</td>
<td></td>
<td>4</td>
<td>8</td>
<td>3</td>
<td>6.75</td>
</tr>
<tr>
<td>Minimum spacing</td>
<td>(s_{min})</td>
<td>[in]</td>
<td></td>
<td>2.5</td>
<td>2.75</td>
<td>3.5</td>
<td>4</td>
</tr>
</tbody>
</table>

\(^{(1)}\) Before and after setting \(^{(2)}\) refers to top of shell was flush with concrete surface.

3.5 Characterization of load-displacement curves

The following section discusses the terminology issued to characterize anchor behavior as observed from the load-displacement relationship. Characterization occurs at the individual test level and also as an aggregate of all tests performed. To differentiate results from an individual test and the mean results from several tests, the following terminology is used. The peak load achieved during testing is termed the ultimate load \(N_u\). The ultimate load from test 1 is \(N_{u1}\), the ultimate load from test 2 is \(N_{u2}\) and so on to the number of tests \(N_{un}\). The mean ultimate load \(N_{um}\) from a series of \(n\) tests is determined as the average of the ultimate loads. Similar subscripts are used for ultimate displacement \(\delta_u\) and mean ultimate displacement \(\delta_{um}\) as shown in the equations below. The ultimate displacement \(\delta_u\) is characterized as the displacement at ultimate load \(N_u\). Subscripts “uncr” and “cr” are added to
denote the mean ultimate strength in uncracked concrete $N_{um,uncr}$ and the mean ultimate strength in cracked concrete $N_{um,cr}$.

\[
Mean \; ultimate \; load \quad N_{um} = \frac{\sum_{i=1}^{n} N_{ui}}{n} \quad 3-1
\]

\[
Mean \; ultimate \; displacement \quad \delta_{um} = \frac{\sum_{i=1}^{n} \delta_{ui}}{n} \quad 3-2
\]

Figure 3.13 describes the key parameters that are used to quantify anchor axial tension load-displacement relationships. The three key parameters are load ($N$), displacement ($\delta$), and secant stiffness ($K$). The parameters are calculated at several key points along the $N-\delta$ curve as shown in Figure 3.13 as a) initial load (50%$N_u$), b) ultimate load and c) post peak load. The load-displacement curve could be from an individual test, in which case the ultimate load is $N_u$, or the load-displacement curve could be the mean curve from several tests, $N_{um}$. 
The ultimate load $N_u$ (point B) is the maximum axial load achieved by the anchor during the test and the corresponding displacement at ultimate load ($\delta_u$). The initial stiffness $K_i$ (point A) is found by taking the load at 50% of ultimate ($0.5N_u$) and the corresponding displacement $\delta_i$. The initial stiffness is the secant stiffness resulting from the following equation:

$$K_i = \frac{0.5N_u}{\delta_i}$$

The post peak load $N_{pp}$ (point C) and the corresponding post peak displacement ($\delta_{pp}$) are a measure of the anchors final usable load and displacement. The commonly used definition of post peak load is to use a percentage of $N_u$ after the peak load, typically 85% for
seismic applications. This definition for $N_{pp}$ is used for the purpose of this dissertation and $\delta_{pp}$ is the corresponding displacement at $N_{pp}$:

$$N_{pp} = 0.85 N_u$$

3.6 Results

The complete test results and data sheets are provided by Watkins and Hutchinson (2010a and 2010b). This section summarizes selected key results. Table 3.3 provides a summary of the ultimate loads $N_u$. Table 3.4 summarizes the anchor axial displacement at ultimate load ($\delta_u$).

<table>
<thead>
<tr>
<th>Anchor ID</th>
<th>Anchor Type</th>
<th>Crack Width [in [mm]]</th>
<th>Ultimate Tension Load, $N_u$ [kips]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Test 1</td>
<td>Test 2</td>
</tr>
<tr>
<td>N4</td>
<td>Epoxy</td>
<td>0.00 [0.0]</td>
<td>8.670</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.03 [0.8]</td>
<td>4.569</td>
</tr>
<tr>
<td>N5</td>
<td>Expansion</td>
<td>0.00 [0.0]</td>
<td>7.948</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.02 [0.5]</td>
<td>8.284</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.03 [0.8]</td>
<td>6.388</td>
</tr>
<tr>
<td>N7</td>
<td>Drop-In</td>
<td>0.00 [0.0]</td>
<td>5.973</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.03 [0.8]</td>
<td>2.830</td>
</tr>
<tr>
<td>N9</td>
<td>Undercut</td>
<td>0.00 [0.0]</td>
<td>11.203</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.03 [0.8]</td>
<td>5.809</td>
</tr>
<tr>
<td>Anchor ID</td>
<td>Anchor Type</td>
<td>Crack Width in [mm]</td>
<td>Displacement at ultimate load, ( \delta_u ) [inch]</td>
</tr>
<tr>
<td>-----------</td>
<td>-------------</td>
<td>---------------------</td>
<td>-------------------------------------------------</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Test 1</td>
<td>Test 2</td>
</tr>
<tr>
<td>N4</td>
<td>Epoxy</td>
<td>0.00 [0.0]</td>
<td>0.024</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.03 [0.8]</td>
<td>0.040</td>
</tr>
<tr>
<td>N5</td>
<td>Expansion</td>
<td>0.00 [0.0]</td>
<td>0.523</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.02 [0.5]</td>
<td>0.600</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.03 [0.8]</td>
<td>0.550</td>
</tr>
<tr>
<td>N7</td>
<td>Drop-In</td>
<td>0.00 [0.0]</td>
<td>0.020</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.03 [0.8]</td>
<td>0.251</td>
</tr>
<tr>
<td>N9</td>
<td>Undercut</td>
<td>0.00 [0.0]</td>
<td>0.119</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.03 [0.8]</td>
<td>0.089</td>
</tr>
</tbody>
</table>
Table 3.5 summarizes the key monotonic tension test results in terms of load, displacement, and stiffness.

### Table 3.5 Mean monotonic tension test results

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Symbol</th>
<th>Units</th>
<th>Anchor ID</th>
<th>Symbol</th>
<th>Units</th>
<th>Anchor Type</th>
<th>Parameter</th>
<th>Symbol</th>
<th>Units</th>
<th>Anchor ID</th>
</tr>
</thead>
<tbody>
<tr>
<td>Anchor ID</td>
<td></td>
<td></td>
<td></td>
<td>Anchor</td>
<td>Type</td>
<td>Crack width</td>
<td>Crack width</td>
<td>w</td>
<td>[mm]</td>
<td>N4</td>
</tr>
<tr>
<td>Anchor Type</td>
<td></td>
<td></td>
<td>N5 Expansion</td>
<td></td>
<td></td>
<td></td>
<td>Ultimate load</td>
<td>N&lt;sub&gt;u,uncr&lt;/sub&gt;</td>
<td>[kip]</td>
<td>8.860</td>
</tr>
<tr>
<td>Crack width</td>
<td>w</td>
<td>[mm]</td>
<td>N5 Expansion</td>
<td></td>
<td></td>
<td></td>
<td>Ultimate load</td>
<td>N&lt;sub&gt;u,cr&lt;/sub&gt;</td>
<td>[kip]</td>
<td>4.353</td>
</tr>
<tr>
<td></td>
<td>w</td>
<td>[in]</td>
<td>N5 Drop-In</td>
<td></td>
<td></td>
<td></td>
<td>N&lt;sub&gt;u,uncr&lt;/sub&gt; / N&lt;sub&gt;u,cr&lt;/sub&gt;</td>
<td>unitless</td>
<td>0.491</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>N9 Undercut</td>
<td></td>
<td></td>
<td></td>
<td>Ultimate displacement</td>
<td>δ&lt;sub&gt;u,uncr&lt;/sub&gt;</td>
<td>[in]</td>
<td>0.025</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Ultimate displacement</td>
<td>δ&lt;sub&gt;u,cr&lt;/sub&gt;</td>
<td>[in]</td>
<td>0.043</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Pre-peak displacement at 50% of ultimate load</td>
<td>δ&lt;sub&gt;i,uncr&lt;/sub&gt;</td>
<td>[in]</td>
<td>0.009</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Pre-peak displacement at 50% of ultimate load</td>
<td>δ&lt;sub&gt;i,cr&lt;/sub&gt;</td>
<td>[in]</td>
<td>0.013</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Post-peak displacement at 85% of ultimate load</td>
<td>δ&lt;sub&gt;pp,uncr&lt;/sub&gt;</td>
<td>[in]</td>
<td>0.060</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Post-peak displacement at 85% of ultimate load</td>
<td>δ&lt;sub&gt;pp,cr&lt;/sub&gt;</td>
<td>[in]</td>
<td>0.065</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Initial stiffness, uncracked</td>
<td>K&lt;sub&gt;i,uncr&lt;/sub&gt;</td>
<td>[kip/in]</td>
<td>492</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Initial stiffness, cracked</td>
<td>K&lt;sub&gt;i,cr&lt;/sub&gt;</td>
<td>[kip/in]</td>
<td>167</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Secant stiffness at ultimate load, uncracked</td>
<td>K&lt;sub&gt;s,uncr&lt;/sub&gt;</td>
<td>[kip/in]</td>
<td>354</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Secant stiffness at ultimate load, cracked</td>
<td>K&lt;sub&gt;s,cr&lt;/sub&gt;</td>
<td>[kip/in]</td>
<td>101</td>
</tr>
</tbody>
</table>
Figure 3.14 provides a graphical summary of the mean, minimum and maximum ultimate loads for anchors in cracked and uncracked concrete. For comparison, the test results from a companion test program conducted at San Diego State University (SDSU) are shown for epoxy and expansion anchors [Hoehler, 2009]. It is noted that the effective embedment varied for the different anchor types. All anchors experienced a reduction in ultimate strength when located in a crack.

The mean ultimate displacement capacity of the anchors was compared in Figure 3.15. In uncracked concrete, anchors epoxy and drop-in anchors exhibited ultimate displacements less than 0.05 inches [1 mm]. The undercut anchor exhibited a slightly larger ultimate displacement in uncracked concrete, less than 0.16 inches [4 mm]. The expansion anchor showed the largest displacement at ultimate in uncracked concrete of 0.56 inches [14 mm].
Figure 3.15 also shows the scatter in the mean ultimate displacement results by displaying the maximum and minimum values obtained during testing. In general, there was less scatter in the ultimate displacement for anchors failing by concrete breakout and more scatter for pull-through failure.

![Figure 3.15 Mean ultimate displacement, $\delta_{um}$](image)

The trends in ultimate displacement were similar in cracked concrete as well, with the exception that the drop-in anchor, which exhibited a larger displacement in cracked concrete. The epoxy anchor exhibited the smallest ultimate displacement in cracked concrete, less than 0.05 inches [1 mm]. The undercut anchor, which also failed by concrete breakout had a slightly higher ultimate displacement in uncracked concrete, 0.11 inches [2.8 mm], followed by the drop-in anchor which had an ultimate displacement of 0.23 inches [5.8 mm]. The
expansion anchor, had the highest displacement at ultimate in cracked concrete of 0.50 inches [13 mm].

Figure 3.16 summarizes the axial tension load-displacement behavior for epoxy, expansion, drop-in, and undercut anchors in uncracked and cracked concrete with 0.03 inches [0.8 mm] wide cracks. In general, the axial strength and stiffness decrease when the anchor is located in a crack. Additionally the failure modes of the anchors vary by anchor type and load resisting mechanism. The following abbreviations are used for failure modes:

**Abbreviations for typical anchor failure modes:**

- **C** – Concrete breakout failure
- **S** – Steel failure
- **Po** – Pullout (including sleeve)
- **Pt** – Pull-through (sleeve stays in hole)
- **Sp** – Splitting failure
Figure 3.16 Axial tension load-displacement behavior and failure mode

Legend: C=Concrete, S=Steel, Po=Pullout (incl. sleeve), Pt=Pull-through
3.6.1 Failure Photographs

Photographs of the failure modes for all anchor types are provided in Figure 3.17 to Figure 3.24. Failure photographs are grouped by anchor type and by uncracked and cracked concrete.

Figure 3.17 Epoxy anchor failure photos in uncracked concrete

Figure 3.18 Epoxy anchor failure photos in cracked concrete (w=0.8 mm)

Figure 3.19 Expansion anchor failure photos in uncracked concrete

Note: several photos have been edited with grey boxes to remove the identity of the product
Figure 3.20 Expansion anchor failure photos in cracked concrete ($w=0.8$ mm)

Figure 3.21 Drop-in anchor failure photos in uncracked concrete

Figure 3.22 Drop-in anchor failure photos in cracked concrete ($w=0.8$ mm)

Note: several photos have been edited with grey boxes to remove the identity of the product
3.6.2 Mean Load Displacement Relationships

Mean load-displacement curves are computed for each test series by taking the mean load of the three (or four) individual test load displacement test curves at 0.05 inch displacement intervals (Figure 3.25 and Figure 3.26). In general, in both cracked and
uncracked concrete, the anchors with breakout failure epoxy, drop-in, and undercut exhibit less displacement capacity than the expansion anchor. It is noted that little difference is observed for the expansion anchor if placed in 0.5 or 0.8 mm [0.02 or 0.03 inch] static cracks (Figure 3.26)
Figure 3.25 Mean load versus displacement curves, uncracked concrete

Figure 3.26 Mean load versus displacement curves, cracked concrete
Figure 3.27 Cracked strength reduction factor comparison with ACI 318, (2008) code and results from Eligehausen and Balogh, (1995)

Figure 3.27 compares the $\Lambda$ factor (ratio of $N_{um,cr}$ to $N_{um,uncr}$) obtained from testing with both design code values and the literature database of Eligehausen and Balogh (1995). The design code values are calculated from ACI 318, 2008 based on product specific design recommendations from the ICC-ESR reports. The design code $\Lambda$ factor matches the test results well for the epoxy, concrete breakout anchor. The code gives a higher $\Lambda$ factor for all other anchors when compared to test results. The low $\Lambda$ factor for the undercut anchor of 0.55 is due to a high ultimate capacity in uncracked concrete rather than a low ultimate capacity in cracked concrete. When the cracked capacity is compared with the standard breakout equation with a mean theoretical $k$ factor of 35, the $\Lambda$ factor changes from 0.55 to 0.73, which is more consistent with historical results of 0.7 for undercut anchors and headed studs.
3.7 Acknowledgement of Publications

For completeness of the present dissertation, a summary of data from Watkins and Hutchinson (2010a,b), in which the author of this dissertation was the primary author, is reproduced in this chapter.


3.8 Summary Remarks

- Anchor behavior has been characterized by the tension-load displacement curves and key features of the $N$-$\delta$ curves have been extracted and compared for the various anchor types. The key features being the shape of the $N$-$\delta$ curve, load ($N$), displacement ($\delta$), and stiffness ($K$) at the following significant points: ultimate load, initial load (i.e. pre-peak load equal to 50% of ultimate load), and post-peak load equal to 85% of ultimate load.

- Anchor ultimate axial tension load was found to be a more dependable and reproducible performance parameter than anchor ultimate axial displacement. The
variability of ultimate load was much smaller than the variability of ultimate displacement.

- Anchor axial stiffness is a highly variable parameter; however, it was observed that for a given anchor, the stiffness in cracked concrete is always less than the stiffness in uncracked concrete. This trend is true for both initial secant stiffness ($K_i$) and secant stiffness at ultimate load ($K_u$).

- Anchors with concrete breakout failure modes were found to have higher stiffness than anchors with pull-through failure for both initial secant stiffness ($K_i$) and secant stiffness at ultimate load ($K_u$).

- Anchors with concrete breakout failure modes, epoxy, drop-in, and undercut were found to have smaller ultimate displacements than anchors with pull-through failure for expansion. Therefore, if a designer wants to limit anchor ultimate displacements, such as for displacement sensitive components, an anchor with concrete breakout failure mode could be selected. However, if a designer wants a large anchor displacement capacity, for example load redistribution or energy absorption capacity, then a pull-through type anchor such as expansion anchor could be selected.

- An expansion anchor that is designed to have a pull-through failure mode can produce a reliable load plateau and large displacement capacity in both uncracked and cracked concrete.

- For anchors designed for use in cracked concrete (epoxy, expansion, and undercut), the presence of cracks was not found to significantly affect the mean ultimate displacement ($\delta_{um}$).
Chapter 4 Shake Table Test Program

4.1 Introduction

This chapter presents the experimental shake table test program of model NCSs anchored in cyclic cracked concrete. A detailed description of the test matrix, test naming system, specimens (cracked concrete slabs and anchors), instrumentation, test setup, test procedures/protocols, test motions, test sequence and data post-processing techniques are provided. The goals of the testing program were as follows:

- Document anchored NCS response under varied input levels of earthquake shaking
- Obtain simultaneous measurements of input motion, NCS response, anchor load, anchor displacement, and crack width due to seismic demand
- Determine the influence of anchor behavior on NCS behavior
- Characterize the contribution of anchor stiffness to the total NCS system stiffness
- Investigate the influence of loss of preload, anchor ductility and anchor deformation capacity on NCS response
- Demonstrate or disprove correlation between anchor seismic approval and anchor performance during shake table tests of an anchored NCS
- Extract instantaneous anchor tension loads and corresponding crack width and determine the correlation, if any, between anchor load and simultaneous crack width
4.2 Test Matrix

The dynamic testing program consisted of six main parts: 1) WALLE system identification, 2) Variable phase correlation tests, 3) Motion repeatability, 4) Pre-failure tests, 5) Failure tests, and 6) In-phase correlation tests. The dynamic test matrix is summarized in Table 4.1. A total of 188 tests were conducted. Table 4.2 describes the test naming system used for all WALLE dynamic tests. The test naming provides information for each test regarding the following test properties: (1) version of the test, (2) test configuration, (3) anchor type, (4) crack record and target maximum crack width, (5) input motion type, (6) and motion scale. The impact hammer tests, which were run as part of the WALLE system identification are listed in Table 4.3 as these tests used a separate data acquisition system as well as specialized commercially available data processing software. Table 4.4 lists all of the individual tests, which make up the WALLE dynamic test series. The test series are described in more detail in the following sections.
Table 4.1 Dynamic test program

<table>
<thead>
<tr>
<th>Test Series</th>
<th>No. Tests</th>
<th>Test Description</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>WALLE System identification</strong></td>
<td>15</td>
<td><strong>Impact hammer</strong> characterization tests</td>
</tr>
<tr>
<td></td>
<td>30</td>
<td><strong>White noise</strong> characterization tests (uncracked and $w=0.8$ mm static cracks)</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td><strong>Unanchored Rocking</strong> characterization tests</td>
</tr>
<tr>
<td><strong>Variable phase correlation tests</strong></td>
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<td>Version 1: Variable phase correlation tests with <strong>flexible WALLE &amp; undercut anchors</strong></td>
</tr>
<tr>
<td></td>
<td>20</td>
<td>Version 1: Variable phase correlation tests with <strong>stiff WALLE &amp; undercut anchors</strong></td>
</tr>
<tr>
<td></td>
<td>21</td>
<td>Version 2: Variable phase correlation tests with <strong>flexible WALLE &amp; undercut anchors</strong></td>
</tr>
<tr>
<td></td>
<td>20</td>
<td>Version 2: Variable phase correlation tests with <strong>stiff WALLE &amp; undercut anchors</strong></td>
</tr>
<tr>
<td><strong>Pre-failure tests</strong></td>
<td>22</td>
<td><strong>Pre-failure tests</strong> with anchors (Epoxy, expansion, drop-in, &amp; undercut anchors)</td>
</tr>
<tr>
<td><strong>Failure tests</strong></td>
<td>7</td>
<td><strong>Failure tests</strong> with anchors (Epoxy, expansion, drop-in, &amp; undercut anchors)</td>
</tr>
<tr>
<td><strong>In-phase correlation</strong></td>
<td>10</td>
<td><strong>In-phase correlation</strong> tests (Structural tests)</td>
</tr>
<tr>
<td><strong>Motion repeatability</strong></td>
<td>18</td>
<td><strong>Shake table motion</strong> characterization tests</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td><strong>Crack record</strong> characterization motions</td>
</tr>
<tr>
<td></td>
<td><strong>194</strong></td>
<td><strong>TOTAL TESTS</strong></td>
</tr>
</tbody>
</table>

Note: 6 additional WALLE dynamic tests were conducted at SDSU (3 tests on epoxy anchors and 3 tests of expansion anchors). Data from those tests are presented later as a comparison for anchor performance in uncracked concrete.
### Table 4.2 Test naming system

<table>
<thead>
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<th>Test No.</th>
<th>Test Name</th>
<th>Date</th>
</tr>
</thead>
<tbody>
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<td>WALLE.R N9 Hammer_1000lb</td>
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<td>H2</td>
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<td>10/30/09</td>
</tr>
<tr>
<td>H4</td>
<td>WALLE.F N9 Hammer2 50lb</td>
<td></td>
</tr>
<tr>
<td>H5</td>
<td>WALLE.F N9 Hammer_50lb_b</td>
<td>10/30/09</td>
</tr>
<tr>
<td>H6</td>
<td>WALLE.F N5 Hammer_2500lb</td>
<td>11/04/09</td>
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<td>H7</td>
<td>WALLE.F N5 Hammer_0lb</td>
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<td>H15</td>
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<td>01/27/10</td>
</tr>
</tbody>
</table>

**Legend:**
- **(1)** Version
  - Blank = V1
  - V2 = Version2
- **(2)** Configuration
  - WALLE.F = Flexible
  - WALLE.R = Stiff
  - STRUCT = In-Phase
  - TABLE = Table only (no crack)
  - CRACK = Crack only (no table)
- **(3)** Anchor Type
  - N4 = epoxy
  - N5 = expansion
  - N7 = drop-in
  - N9 = undercut
- **(4)** Max Crack Width
  - UC = Uncracked
  - CC05 = 0.5mm
  - CC08 = 0.8mm
  - CP1 = Crack Protocol (1Hz)
  - CP2 = Crack Protocol (2Hz)
  - CR14 = Crack Record (FM14)
- **(5)** Input Motion
  - WN = White Noise
  - FMxx = Floor Motion 01 to 20
  - D1 = 1Hz WALLE displacement*
  - D14 = FM14 WALLE displacement*
  - *STRUCT tests only
- **(6)** Motion Scale
  - 20P = 20%
  - 20Pa = re-run of 20%

### Table 4.3 Impact hammer system identification test series

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<th>Test Name</th>
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Table 4.4 Test matrix WALLE dynamic (continued)
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Test name
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V2.WALLE.F.N9.CC08.FM01
V2.WALLE.F.N9.CC08.FM11
V2.WALLE.F.N9.CC08.FM06
V2.WALLE.F.N9.CC08.FM19
V2.WALLE.F.N9.CC08.FM14
V2.WALLE.F.N9.CC08.FM16
V2.WALLE.F.N9.CC08.FM04
V2.WALLE.F.N9.CC08.FM10
V2.WALLE.F.N9.CC08.FM17
V2.WALLE.F.N9.CC08.FM02
V2.WALLE.F.N9.CC08.FM18
V2.WALLE.F.N9.CC08.FM12
V2.WALLE.F.N9.CC08.FM08
V2.WALLE.F.N9.CC08.FM03
V2.WALLE.F.N9.CC08.FM20.36P
V2.WALLE.F.N9.CC08.FM20
V2.WALLE.F.UA.CC08.FM05
V2.WALLE.F.UA.CC08.FM13
V2.WALLE.F.UA.CC08.FM04
V2.WALLE.F.UA.CC08.FM10
V2.WALLE.F.UA.CC08.FM03
V2.WALLE.F.UA.CC08.FM20
V2.WALLE.F.N9.UC.WN2
V2.WALLE.F.N9.CC08.WN2
V2.WALLE.F.N9.UC.WN3
V2.WALLE.F.N9.CC08.WN3
V2.WALLE.F.N9.CC08.FM02.20P
V2.WALLE.F.N9.CC08.FM02.40P
V2.WALLE.F.N9.CC08.FM02.60P

Version

Test Series

WALLE
Setup

Version 2
Version 2
Version 2
Version 2
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Version 2
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Sys ID
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Sys ID
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Failure
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Failure

Rigid
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Flexible
Flexible
Flexible
Flexible
Flexible
Flexible
Flexible
Flexible

Maximum
Input Motion
Target
Building
Record
Crack
Width (mm)
0
0.8
0.8
0.8
0.8
0.8
0.8
0.8
0.8
0.8
0.8
0.8
0.8
0.8
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0
0.8
0.8
0.8
0.8

WhiteNoise
WhiteNoise
baja
super
irpina
nga
irpina
baja
kobe00
friuli
super
sanf
kobe00
super
super
nga
WhiteNoise
WhiteNoise
WhiteNoise
kobe00
kobe00
friuli
super
north
north
WhiteNoise
WhiteNoise
WhiteNoise
WhiteNoise
super
irpina
super
baja
friuli
irpina
baja
nga
super
super
kobe00
sanf
friuli
nga
kobe00
kobe00
kobe00
north
north
super
super
super
super
sanf
friuli
north
super
WhiteNoise
WhiteNoise
WhiteNoise
WhiteNoise
kobe00
kobe00
kobe00

n/a
n/a
8st
20st
8st
20st
2st
12st
8st
8st
4st
2st
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12st
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2st
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4st
12st
2st
8st
2st
20st
n/a
n/a
n/a
n/a
2st
2st
2st

Floor

Target
Scale %

n/a
n/a
5
18
1
4
1
6
5
3
1
1
4
9
2
1
n/a
n/a
n/a
10
1
2
5
1
2
n/a
n/a
n/a
n/a
1
1
2
6
2
1
5
1
9
5
10
1
3
4
1
4
5
2
1
18
18
1
2
1
3
1
18
n/a
n/a
n/a
n/a
1
1
1

100
100
27
36
45
60
24
75
72
30
30
32
90
75
28
60
100
100
100
120
36
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87
23
70
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18
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23
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6
36
81
15
10
8
10
6
81
100
100
100
100
20
40
60


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<th>Maximum Target Crack Width (mm)</th>
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<th>Building</th>
<th>Floor</th>
<th>Target Scale %</th>
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<td>In-Phase</td>
<td>0.8</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td></td>
</tr>
<tr>
<td>167</td>
<td>WALLE.F.N4.UC.10.1</td>
<td>Version 1</td>
<td>Failure SDSU Flexible</td>
<td>0</td>
<td>kobe00</td>
<td>2st</td>
<td>1</td>
<td>10</td>
<td></td>
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<tr>
<td>168</td>
<td>WALLE.F.N4.UC.20.1</td>
<td>Version 1</td>
<td>Failure SDSU Flexible</td>
<td>0</td>
<td>kobe00</td>
<td>2st</td>
<td>1</td>
<td>20</td>
<td></td>
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<td>Version 1</td>
<td>Failure SDSU Flexible</td>
<td>0</td>
<td>kobe00</td>
<td>2st</td>
<td>1</td>
<td>30</td>
<td></td>
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<tr>
<td>170</td>
<td>WALLE.F.N5.UC.20.1</td>
<td>Version 1</td>
<td>Failure SDSU Flexible</td>
<td>0</td>
<td>kobe00</td>
<td>2st</td>
<td>1</td>
<td>20</td>
<td></td>
</tr>
<tr>
<td>171</td>
<td>WALLE.F.N5.UC.100.1</td>
<td>Version 1</td>
<td>Failure SDSU Flexible</td>
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<td>kobe00</td>
<td>2st</td>
<td>1</td>
<td>100</td>
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<tr>
<td>172</td>
<td>WALLE.F.N5.UC.150.1</td>
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<td>Failure SDSU Flexible</td>
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<td>kobe00</td>
<td>2st</td>
<td>1</td>
<td>150</td>
<td></td>
</tr>
</tbody>
</table>
4.3 Instrumentation

Instrumentation for the WALLE-CCILR tests consisted of 83 total channels: 17 acceleration, 28 displacements, 10 load, and 28 strain gases. Instrumentation was designed and placed to monitor both the WALLE anchorage system as well as the CCILR and shake table input systems. Two data acquisition systems were used, herein called the SDSU DAQ and the UCSD DAQ. A complete list of channels recorded by the SDSU DAQ is given in Table 4.5. A complete list of channels recorded by the UCSD DAQ is given in Table 4.6. The channel “No.” indicates the column in the data file where data for that channel is stored.

The SDSU DAQ consisted of a DaqBook2000, DBK41, DBK4, DBK32A accelerometer unit, DBK65 transducer module, DBK48+ four 8B signal conditioning modules, and three DBK43B signal conditioning modules. Table 4.5 and Table 4.6 also provide reference to the instrument name, which are referenced in Figure 4.2. Figure 4.1 shows a plan view of the WALLE instrumentation load, displacement and strain channels. Figure 4.2 provides the locations of all WALLE accelerometers as well as the elevation of the WALLE center of gravity (CG) for the flexible and stiff configurations. Figure 4.3 depicts all of the instrumentation used to monitor the CCILR. Figure 4.4 shows the CCILR instrumentation for the in-phase correlation test series (STRUCT) showing the added instrumentation of actuator 3 (Act3) and noting that the table was held fixed during these tests. A photograph of the overall anchor instrumentation and closeup of the instrumentation for the Southwest anchor are shown in Figure 4.5. The key measurements depicted are anchor load (LW01 to LW04) and anchor displacement (DispN1 to N4). Later in the analysis sections, anchor load is referred to as $N$ and anchor axial displacement as $\delta$ and the anchors are referred to by Cardinal positions, NW, NE, SW, SE.
<table>
<thead>
<tr>
<th>No.</th>
<th>Name</th>
<th>Units</th>
<th>Description and Serial Number if applicable</th>
<th>Instrument Type / Manufacturer</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Time</td>
<td>sec</td>
<td>Time vector</td>
<td>50g accel. Dytran 3059A</td>
</tr>
<tr>
<td>2</td>
<td>Accl1</td>
<td>g</td>
<td>Accel. 1 (960)</td>
<td>50g accel. Dytran 3059A</td>
</tr>
<tr>
<td>3</td>
<td>Accl2</td>
<td>g</td>
<td>Accel. 2 (1386)</td>
<td>50g accel. Dytran 3059A</td>
</tr>
<tr>
<td>4</td>
<td>Accl3</td>
<td>g</td>
<td>Accel. 3 (1733)</td>
<td>50g accel. Dytran 3059A</td>
</tr>
<tr>
<td>5</td>
<td>Accl4</td>
<td>g</td>
<td>Accel. 4 (1734)</td>
<td>50g accel. Dytran 3059A</td>
</tr>
<tr>
<td>6</td>
<td>Accl5</td>
<td>g</td>
<td>Accel. 5 (1735)</td>
<td>50g accel. Dytran 3059A</td>
</tr>
<tr>
<td>7</td>
<td>Accl6</td>
<td>g</td>
<td>Accel. 6 (1736)</td>
<td>50g accel. Dytran 3059A</td>
</tr>
<tr>
<td>8</td>
<td>Accl7</td>
<td>g</td>
<td>Accel. 7 (1737)</td>
<td>50g accel. Dytran 3059A</td>
</tr>
<tr>
<td>9</td>
<td>Accl8</td>
<td>g</td>
<td>Accel. 8 (1738)</td>
<td>50g accel. Dytran 3059A</td>
</tr>
<tr>
<td>10</td>
<td>DispN1</td>
<td>in</td>
<td>Anchor axial disp. N1 (119587)</td>
<td>LVDT (4&quot;) RDP DCTH2000C</td>
</tr>
<tr>
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<td>DispN2</td>
<td>in</td>
<td>Anchor axial disp. N2 (119030)</td>
<td>LVDT (4&quot;) RDP DCTH2000C</td>
</tr>
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<td>DispN3</td>
<td>in</td>
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<td>LVDT (4&quot;) RDP DCTH2000C</td>
</tr>
<tr>
<td>13</td>
<td>DispN4</td>
<td>in</td>
<td>Anchor axial disp. N4 (122033)</td>
<td>LVDT (4&quot;) RDP DCTH2000C</td>
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<tr>
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<td>DispV1</td>
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<td>WALLE disp in dir. of shaking, base (right)</td>
<td>String Pot (15&quot;) Ametek P-15A-5K(HD)</td>
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<tr>
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<td>in</td>
<td>WALLE disp in dir. of shaking, base (left)</td>
<td>String Pot (15&quot;) Ametek P-15A-5K(HD)</td>
</tr>
<tr>
<td>16</td>
<td>DispV3</td>
<td>in</td>
<td>WALLE disp perp. to dir. of shaking at base</td>
<td>not used @ UCSD</td>
</tr>
<tr>
<td>17</td>
<td>DispLon</td>
<td>in</td>
<td>WALLE disp in dir. of shaking at 48&quot;</td>
<td>String Pot (50&quot;) Ametek P-50A-5K(HD)</td>
</tr>
<tr>
<td>18</td>
<td>DispTra</td>
<td>in</td>
<td>WALLE disp perp. to dir. of shaking at 51.5&quot;</td>
<td>String Pot (25&quot;) Ametek P-25A-5K(HD)</td>
</tr>
<tr>
<td>19</td>
<td>DispTab</td>
<td>in</td>
<td>Table platen disp in dir. of shaking</td>
<td></td>
</tr>
<tr>
<td>20</td>
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<td>21</td>
<td>SAMU1A</td>
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<td>SAMU pin reading</td>
<td>custom</td>
</tr>
<tr>
<td>22</td>
<td>SAMU1B</td>
<td>m/V</td>
<td>SAMU pin reading</td>
<td>custom</td>
</tr>
<tr>
<td>23</td>
<td>SAMU1C</td>
<td>m/V</td>
<td>SAMU pin reading</td>
<td>custom</td>
</tr>
<tr>
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<td>SAMU1D</td>
<td>m/V</td>
<td>SAMU pin reading</td>
<td>custom</td>
</tr>
<tr>
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<td>SAMU2A</td>
<td>m/V</td>
<td>SAMU pin reading</td>
<td>custom</td>
</tr>
<tr>
<td>26</td>
<td>SAMU2B</td>
<td>m/V</td>
<td>SAMU pin reading</td>
<td>custom</td>
</tr>
<tr>
<td>27</td>
<td>SAMU2C</td>
<td>m/V</td>
<td>SAMU pin reading</td>
<td>custom</td>
</tr>
<tr>
<td>28</td>
<td>SAMU2D</td>
<td>m/V</td>
<td>SAMU pin reading</td>
<td>custom</td>
</tr>
<tr>
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</tr>
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<td>m/V</td>
<td>SAMU pin reading</td>
<td>custom</td>
</tr>
<tr>
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<td>m/V</td>
<td>SAMU pin reading</td>
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</tr>
<tr>
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<td>SAMU pin reading</td>
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</tr>
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<td>m/V</td>
<td>SAMU pin reading</td>
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</tr>
<tr>
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<td>m/V</td>
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<td>custom</td>
</tr>
<tr>
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<td>m/V</td>
<td>SAMU pin reading</td>
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</tr>
<tr>
<td>36</td>
<td>SAMU4D</td>
<td>m/V</td>
<td>SAMU pin reading</td>
<td>custom</td>
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<tr>
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<td>Instrumented thread rod 1</td>
<td>not used @ UCSD</td>
</tr>
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<td>Rod2</td>
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<td>Rod3</td>
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<td>lbs</td>
<td>Anchor load washer 1 (229931)</td>
<td>TransducerTech 10k load washer</td>
</tr>
<tr>
<td>42</td>
<td>LW2</td>
<td>lbs</td>
<td>Anchor load washer 2 (239833)</td>
<td>TransducerTech 10k load washer</td>
</tr>
<tr>
<td>43</td>
<td>LW3</td>
<td>lbs</td>
<td>Anchor load washer 3 (239834)</td>
<td>TransducerTech 10k load washer</td>
</tr>
<tr>
<td>44</td>
<td>LW4</td>
<td>lbs</td>
<td>Anchor load washer 4 (239835)</td>
<td>TransducerTech 10k load washer</td>
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### Table 4.6 Instrumentation list (UCSD DAQ)

<table>
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<tr>
<th>No.</th>
<th>Name</th>
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<th>Description</th>
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</thead>
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<tr>
<td>45</td>
<td>TableAccel</td>
<td>g</td>
<td>Accel. N-S underneath table platen</td>
</tr>
<tr>
<td>46</td>
<td>AccelN1</td>
<td>g</td>
<td>Accel. N-S Table East side (A_NS_TBL_E) 4g accelerometer</td>
</tr>
<tr>
<td>47</td>
<td>AccelN2</td>
<td>g</td>
<td>Accel. N-S Table West side (A_NS_TBL_W) 4g accelerometer</td>
</tr>
<tr>
<td>48</td>
<td>AccelN3</td>
<td>g</td>
<td>Accel. N-S Fixed beam East side (A_NS_FE) 4g accelerometer</td>
</tr>
<tr>
<td>49</td>
<td>AccelN4</td>
<td>g</td>
<td>Accel. N-S 4 (A_NS_FW) 4g accelerometer</td>
</tr>
<tr>
<td>50</td>
<td>AccelN5</td>
<td>g</td>
<td>Accel. N-S 5 (A_NS_SLAB_N) 4g accelerometer</td>
</tr>
<tr>
<td>51</td>
<td>AccelV1</td>
<td>g</td>
<td>Accel. Vertical Table North end (A_V_TBL_N) 2g accelerometer</td>
</tr>
<tr>
<td>52</td>
<td>AccelV2</td>
<td>g</td>
<td>Accel. Vertical Table South end (A_V_TBL_S) 2g accelerometer</td>
</tr>
<tr>
<td>53</td>
<td>AccelV3</td>
<td>g</td>
<td>Accel. Vertical Slab CL (A_V_SLAB) 2g accelerometer</td>
</tr>
<tr>
<td>54</td>
<td>TableDisp</td>
<td>in</td>
<td>Table platen disp. in direction of shaking built in to actuator</td>
</tr>
<tr>
<td>55</td>
<td>ActEDisp</td>
<td>in</td>
<td>East CCILR actuator axial disp. built in to actuator</td>
</tr>
<tr>
<td>56</td>
<td>ActWDisp</td>
<td>in</td>
<td>West CCILR actuator axial disp. built in to actuator</td>
</tr>
<tr>
<td>57</td>
<td>AnchorDisp</td>
<td>in</td>
<td>Anchor axial disp., for single anchor reference tests</td>
</tr>
<tr>
<td>58</td>
<td>LP01</td>
<td>mm</td>
<td>Crack width, South crack, East side Lin-Pot (2&quot;) Novotechnik TR10</td>
</tr>
<tr>
<td>59</td>
<td>LP02</td>
<td>mm</td>
<td>Crack width, South crack, West side Lin-Pot (2&quot;) Novotechnik TR10</td>
</tr>
<tr>
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<td>LP03</td>
<td>mm</td>
<td>Crack width, North crack, East side Lin-Pot (2&quot;) Novotechnik TR10</td>
</tr>
<tr>
<td>61</td>
<td>LP04</td>
<td>mm</td>
<td>Crack width, North crack, West side Lin-Pot (2&quot;) Novotechnik TR10</td>
</tr>
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<td>LP05</td>
<td>in</td>
<td>Vertical disp. of sliding end East Lin-Pot (2&quot;) LP 602</td>
</tr>
<tr>
<td>63</td>
<td>LP06</td>
<td>in</td>
<td>Vertical disp. of sliding end West Lin-Pot (2&quot;) LP 602</td>
</tr>
<tr>
<td>64</td>
<td>LP09</td>
<td>in</td>
<td>Relative disp. N-S btw. slab and sliding end East Lin-Pot (2&quot;) LP 602</td>
</tr>
<tr>
<td>65</td>
<td>LP10</td>
<td>in</td>
<td>Relative disp. N-S btw. slab and sliding end West Lin-Pot (2&quot;) LP 602</td>
</tr>
<tr>
<td>66</td>
<td>LP11</td>
<td>in</td>
<td>N-S disp. of sliding beam East Lin-Pot (6&quot;) LP 606</td>
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<tr>
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<td>LP12</td>
<td>in</td>
<td>N-S disp. of sliding beam East Lin-Pot (6&quot;) LP 606</td>
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<tr>
<td>68</td>
<td>LP13</td>
<td>in</td>
<td>Relative disp. N-S btw. CCILR and table platen East Lin-Pot (2&quot;) LP 602</td>
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<td>LP14</td>
<td>in</td>
<td>Relative disp. N-S btw. CCILR and table platen West Lin-Pot (2&quot;) LP 602</td>
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<tr>
<td>70</td>
<td>LP15</td>
<td>in</td>
<td>Relative disp E-W btw. CCILR and table platen South Lin-Pot (2&quot;) LP 602</td>
</tr>
<tr>
<td>71</td>
<td>ActELoad</td>
<td>kips</td>
<td>Actuator load cell CCILR East built in to actuator</td>
</tr>
<tr>
<td>72</td>
<td>ActWLoad</td>
<td>kips</td>
<td>Actuator load cell CCILR West built in to actuator</td>
</tr>
<tr>
<td>73</td>
<td>ActELoadCom</td>
<td>kips</td>
<td>Actuator load command CCILR East --</td>
</tr>
<tr>
<td>74</td>
<td>AnchorLoad</td>
<td>lbs</td>
<td>Axial disp. of anchor (single anchor tests) Load Cell Interface Inc., LW2075-20K</td>
</tr>
<tr>
<td>75</td>
<td>SG01</td>
<td>(\mu)strain</td>
<td>Bar strain (Top bar North-East) FLA5</td>
</tr>
<tr>
<td>76</td>
<td>SG02</td>
<td>(\mu)strain</td>
<td>Bar strain (Bottom bar North-East) FLA5</td>
</tr>
<tr>
<td>77</td>
<td>SG03</td>
<td>(\mu)strain</td>
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<tr>
<td>78</td>
<td>SG04</td>
<td>(\mu)strain</td>
<td>Bar strain (Bottom bar North-West) FLA5</td>
</tr>
<tr>
<td>79</td>
<td>SG05</td>
<td>(\mu)strain</td>
<td>Bar strain (debonded region) FLA5</td>
</tr>
<tr>
<td>80</td>
<td>SG06</td>
<td>(\mu)strain</td>
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<td>Act3Disp</td>
<td>in</td>
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</tr>
<tr>
<td>84</td>
<td>Act3Load</td>
<td>kips</td>
<td>Actuator 3 Load built in to actuator</td>
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</tbody>
</table>
Figure 4.1 WALLE instrumentation: load washer, displacement and strain channels (plan view) – arrows denote displacement measurements.

Figure 4.2 WALLE instrumentation: acceleration channels (arrows denote direction of measurement, open circles are into the page)
Figure 4.3 CCILR instrumentation: plan view and elevation view
Figure 4.4 Additional CCILR instrumentation for in-phase correlation tests elevation view (added instrumentation shown in red)
Figure 4.5 Photograph of anchor instrumentation (a) overview and (b) closeup of the Southwest anchor.
4.5 Test Setup

The following section describes in detail the test setup and instrumentation. For all tests listed as Table 4.1 the setup shown in Figure 2.14 was used, with the exceptions of the in-phase tests, where an additional actuator was attached to WALLE to impose in-phase inertial equivalent loading. Because of the geometry of the slabs, the WALLE could be located in one of three fixed positions for testing in cyclic cracks. The positions are denoted A, B, and C in Figure 4.8 and 4.9. It is noted that the slab was double sided and could therefore be tested on both sides, resulting in a total of six test locations per slab. Cracks generated in the slab are denoted CR1 through CR6 from the south to north end of the slab. Table 4.7 provides a log of WALLE’s position on the slab during each of the test series for each anchor type.

Figure 4.6 WALLE positions A, B, C
It is noted that tests 182 and 185 were conducted at SDSU in uncracked concrete under a separate portion of this research program. Select results from these tests are included in this report for comparison of behavior in uncracked concrete versus cracked concrete.
4.5.1 WALLE White Noise, Correlation, and Failure Test Series

The main test setup for WALLE White Noise, Correlation, and Failure Test Series is shown in Figure 4.8, details of how WALLE is mounted to the slab are provided in Chapter 2 and details of the anchor installation are the same as those for the monotonic tension reference tests as described in Chapter 3. Data sheets describing the WALLE installation details for each test are provided in Appendix C.

![Figure 4.8 Test setup for white noise, variable phase correlation, and failure tests: (a) conceptual sketch and (b) photograph](image)

4.5.2 WALLE Unanchored Rocking Test Setup

The unanchored test setup was the same as described previously for the white noise, correlation, and failure test series except for the following modifications. The tests were conducted with the undercut anchors in place except that the nuts were backed off by 2 inches so that the WALLE was free to rock without tensile restraint from the anchors. The unanchored tests were part of the system identification test series to investigate the frequency and damping characteristics of an unanchored model NCS.
4.5.3 WALLE In-Phase Correlation (Structural) Test Setup

The WALLE-CCILR test setup was modified to allow imposition of inertial equivalent loads in phase with the crack cycling. To accomplish this, the shake table was fixed in displacement control to maintain a zero displacement. WALLE loading was accomplished by adding a 50 kip servo-controlled hydraulic actuator and mounting adapter plate as shown in Figure 4.9 and 4.12. The 50 kip actuator was operated in displacement control.

![Figure 4.9 Photograph of in-phase correlation test setup, isometric view](image_url)
4.5.4 WALLE Impact Hammer Test Setup

Impact hammer testing was conducted on the WALLE to evaluate the model NCSs’ dynamic characteristics (natural frequency and damping). In the impact hammer tests, a lightweight hammer was used to excite the equipment and the response from the excitation is measured by accelerometers attached to the equipment (Figure 4.11).
Attachment of the accelerometers to the equipment was achieved using a magnet mount for Accl2 and by mounting Accl1 directly into the pretapped threaded hole at the top of the WALLE mast. Accelerometers were connected to National Instrument’s data acquisition devices (DAQ): NI 9234, and/or NI cRIO-9233. Both DAQ devices fit into an NI USB 9162 (Hi-Speed USB Carrier) connected to a Dell Laptop installed with ME’Scope, a data acquisition software developed by Vibrant Technology. ME’Scope is a modal analysis software suited for post processing and frequency domain analysis. It is also able to determine modal characteristics. Both the NI 9234 and 9233 contain 4 channels allowing a max of seven accelerometer attachments and one force transducer connection (for the impact hammer). The impact hammer utilized was the PCB 086D05 (mass 1.14 lbs) with a hard rubber impact tip and extender mass.
4.6 Test Procedures / Protocol

4.6.1 WALLE Assembly and Disassembly

The following steps were followed when assembling and disassembling WALLE:

**Assembly of WALLE**

- Start a new data sheet (record all installation parameters)
- Move NCS into position for test (check that there is no debris under the NCS and that it sits flush.)
- Attach catcher system
- Check that pins are centered in SAMU with Pos 1 up
- Attach SAMU and torque 250 ft-lbs (5/8” A490 bolts) (if required)
- Install hold-down blocks at corners of slab
- Attach stopper angles
- Center WALLE (on cracks and E-W)
- Set anchors
  - Measure drill bit cut diameter $d_{cut}$ (document)
  - Set drill depth to desired hole depth (slightly less than desired to account for over drilling)
  - Drill holes into the concrete slab using collars as guides and record hole depth and drill bit diameter on data sheet
  - Install anchor (per manufacturer instructions unless otherwise noted)
  - Clean holes as required by install instructions
  - Check that collar sits flush on concrete (remove excess epoxy as required)
  - Measure anchor height above concrete (document)
- Place collars into SAMUs (collars should fit snug around anchor)
- Using collar setting tool, tighten SAMU pins using SAMU pin tool (socket screwdriver)
- Level WALLE using shims and hot glue shims to slab (small strips of metal)
- Place load washers on anchors and secure with spherical washers and grade 8 nuts and torque anchor to 100% installation torque (if required) reduce torque completely then re-torque to 50% installation torque (if required). If no installation torque is specified, then tighten nuts until anchor tension load is between 50-70lbs (50 lbs target)
- Insert load washer hold down bolts with plastic sleeves
- Install safety stopper angles using small drop-in anchors (1.5” from face of WALLE basetube)
- Attach weight plates (if required)
- Install crack width linear pots LP01-LP04, measuring piston 2” from edge of slab
- Attach string pots
- Install anchor LVDT displacement stands
  - Position stand w/ magnet vertically over anchor head
  - Mark position on concrete w/ pencil
  - Clean contact surfaces
  - Attach using hot melt glue
Hand check all instrumentation channels on data acquisition systems
Warm up pumps (30 minutes)
Pretension anchors as required
Zero displacement offsets in Daqview (as required)
Record anchor pretension
Start recording data

Disassembly of WALLE

Stop hydraulics and secure NCS from tipping
Remove axial anchor LVDTs
Disconnect string pots
Remove weight plates
Remove shackles on primary catcher system
Remove WALLE
Remove shear displacement stoppers
Photograph and record failure observations (place scale in all photos)
Cut off remaining anchors with angle grinder (if required)

4.6.2 WALLE Dynamic Test Sequence

The following steps were followed to prepare for and perform a WALLE dynamic tests. Dynamic tests included white noise tests, variable phase correlation tests, and failure tests. First, the concrete slab and WALLE were prepared for testing to determine that the anchors were located in holes with cracks and record all pertinent information regarding anchor installation. The WALLE data sheets are located in Appendix C.

PREPARE SLAB & WALLE

S1) DRILL AND CLEAN ANCHOR HOLES: using WALLE as a template
S2) OPEN CRACKS: to 0.4 mm (min) 0.8 mm (max)
S3) CHECK CRACKS: with borescope, verify cracks in holes, place a check-mark on slab
S4) INSTALL WALLE: (See WALLE setup/tear down described above)
S5) RECORD MEASUREMENTS: Record on test data sheet all information including: drill bit diameter, hole depth, anchor type, drill type, anchor type, lot number, NCS position, engineer/technician.

BEGINNING OF EACH DAY OF TESTING AND NEW TEST SETUPS

P1) TURN ON SDSU DAQ: and setup laptop and Ethernet cable.

P2) CHECK INSTRUMENTATION: Check WALLE and CCILR instrumentation (beginning of each day of testing, and beginning of new setup positions)

P3) VERIFY SAFETY / CATCHER SYSTEM

P4) CONNECT STRING POTS

P5) PHOTOGRAPH TEST SETUP: include sign with test name in photo

P6) CLOSE CRACKS: Compress slab to 50 kips per actuator in load control and hold for at least 10 seconds

P7) TIGHTEN NUTS ON DYWIDAG BARS: wrench tight condition, check nut tightness approx. every 5 tests

P8) RELEASE LOAD TO ZERO: hairline crack appears slightly visible

STEPS T1-T9: EVERY TEST

T1) MEASURE AND RECORD HAIRLINE CRACK: Optically measure and record the residual crack width

T2) WRENCH TIGHTEN NUTS ON ANCHORS 50lbs target (50lb to 70lb acceptable range)

T3) VERIFY CRACK COMMAND AND TABLE COMMAND HISTORIES PER the TEST MATRIX, also verify shake table limits and settings. (AIC frozen)
T4) PREPARE UCSD DAQ for TEST: Nominal scan rate = 200 samples per second, Length = 300 sec., Save data file as test name from Test Matrix with the suffix “_UCSD”

T5) PREPARE SDSU DAQ for TEST: Save data file as test name from Test Matrix with the suffix “_SDSU”

T6) VERIFY VIDEO SIGN & RECORD TIME OF TEST: verify that sign matches the test name from test matrix

T7) CAMERAS RECORDING WITH SIGN: use sign as first frame of video

T8) START DATA ACQUISITION (UCSD AND SDSU SYSTEMS): Activate warning buzzer, Countdown: 5, 4, 3 (start data scanning), 2, 1 (start table)

T10) RUN TEST

T11) DATA ACQUISITION OFF?

T12) CAMERAS OFF?

T13) SAVE DATA FILE: store in proper directory, transfer to laptop with memory stick

T14) TRANSFER DAT FILES FROM UCSD and SDSU computers to laptop

T15) CHECK DATA: Run Matlab post-processing script and inspect plots of each data channel

T16) DISSASSEMBLE WALLE: (only if end of test series)

T17) POST TEST INSPECTION: Photograph a plan view and any anchor failures. Record observations on the data sheets including dimensions of concrete breakout cones, etc…

4.6.3 White Noise Test Procedure

White noise excitation was used before each test series to assess the dynamic properties of the model NCS-anchor system and to track the progression of damage and
system softening. The input to the shake table consisted of Gaussian white noise with a root-mean-square (RMS) amplitude of 0.03g for a duration of approximately 90 seconds per test. Two white noise runs with varying crack boundary conditions were conducted on each WALLE setup for the four different anchor types tested. The first white noise test, representing an uncracked concrete boundary condition, was performed with anchors installed in hairline cracks that were precompressed closed with a pre-compression of 100 kips, then released to zero force and held constant at the hairline width with the actuators fixed in displacement control. The second white noise test, representing a cracked concrete condition, was performed with cracks opened and held at a constant crack width $\Delta w$ of 0.03” [0.8mm] with the cracking actuators in displacement control.

4.6.4 WALLE In-Phase Test Procedure

In-phase correlation testing was conducted using a) the crack cycling protocol (CP1) developed by Wood et. al (2009) and b) a crack history from earthquake FM14, a motion from the numerical study of Wood et al. (2009), taken from the 5th floor of a 12 story building subjected to the Superstition Hills ground motion. These input min phase with crack width cycling, the load cycling was prescribed at a frequency of 1Hz, while the crack cycling was prescribed at twice this frequency, i.e. at a frequency of 2Hz. In this manner, theoretically the cracks are open at each instance that the anchors on the North or South side achieve a load cycle peak (e.g. Figure 4.12). The amplitude of load and crack width was increased step-wise according to the CP1 pattern (Figure 4.13). To achieve load cycling in the anchors, a displacement history was prescribed to WALLE using a 50 kip servo controlled hydraulic actuator (Act3) located along the WALLE vertical mast (Figure 4.4 and 4.11) with a maximum displacement amplitude of 0.25 inches.
Figure 4.12 Description in-phase crack and load cycling protocol showing NCS displacement, crack width, and anchor load cycles.

Figure 4.13 In-phase crack and load cycling protocol (CP1), Wood et al., (2009)
4.7 Input Motions Selection and Characterization

This section presents the background for the selection of motions used in the testing as well as presents the relevant characteristics of the motions themselves and highlights the target versus achieved motion characteristics.

4.7.1 Background for Input Motion Development

The shake table input motions were developed from the nonlinear building simulations of Wood et al. (2009). The numerical study of Wood et al. (2009) involved conducting nonlinear time history analysis of reinforced concrete (RC) frame-braced buildings with heights 2, 4, 8, 12, and 20 stories. These buildings encompassed a period range of 0.25-2.4 seconds. They were assumed to be founded on a site located in Charter Oaks, California with Site Class C, with the spectral acceleration in the vicinity of the site at short periods ($S_s$) and at a period of one second ($S_1$) conservatively estimated as 2.01 g and 0.61 g, respectively. Using procedures of ASCE 7-05 (2005), a target design acceleration response spectrum was generated considering a return period 475 years, thereby representing a 10% in 50 year probability of exceedance. The design spectral ordinates were as follows: $S_{05} = 1.34$ g and $S_{D1} = 0.53$ g. Time histories for the simulation were selected from the PEER database (PEER-NGA Database website) and were scaled to the design spectrum for the site. Scaling was performed using the Geometric Mean Method, implemented by Huang et al. (2009), which minimizes the sum of the squares error between the design (target) spectral acceleration and the spectral acceleration ordinate of the selected records over a defined broad period range. The resulting spectra using scaled records and target design spectrum are shown in Figure 4.14.
Figure 4.14 Elastic 5% damped pseudo-spectral acceleration for scaled ground motions. Individual motions shown in grey (from Wood et al. 2009)

The shake table testing described in this report required two critical input time histories to the NCS-building experimental sub-assembly (CCILR-WALLE), they are the floor level acceleration time histories and the corresponding flexural crack width time histories occurring in a beam at the given floor level. The floor acceleration histories and flexural crack width time histories serve as the shake table input motions and crack cycling time histories. Watkins et al. (2011) describe in detail the methodology for obtaining crack width and floor acceleration histories from the building simulations of Wood et al. (2009). The methodology is summarized as follows. Curvature histories from just outside of the plastic hinge zone were extracted from these simulation results and normalized to a maximum of 1.0 to represent crack width, which were then scaled to the maximum target crack width, \( w_{\text{max}} \). The maximum crack width for most dynamic tests was selected as 0.8 mm (0.03 inch) to represent a maximum crack width that would occur at the yield curvature of a flexural beam.
that is detailed in accordance with standard code practice (e.g. Hoehler, 2006). Failure tests were also repeated at $w_{max} = 0.5$ mm for the expansion anchor because the expansion anchor pull-through failure is known to be sensitive to crack width because of the reduction in expansion force that occurs during crack opening. The simulation normalized curvature histories were scaled using the empirically derived relationship for the slab to obtain the target maximum crack width. Additionally, floor acceleration histories were extracted from each floor from each building. These simulations were representative of a broad range of building types and earthquake motion characteristics and thus represent expected demands on a broad range of real structures.

4.7.2 Selection of Input Motions

A suite of 20 floor input motions was selected from the Wood et al. (2009) nonlinear building simulation results. The goal of the selection process was to obtain a sampling of motions with a wide variety of characteristics in terms of magnitude, earthquake, distance, frequency content, and effective duration. To obtain motions from each type of building, and hence predominant period of floor motion, it was decided that four motions would be selected from each building height (2, 4, 8, 12, 20 stories). The selection was also limited to the capabilities of the shake table in terms of displacement, velocity, acceleration, and oil flow. The limitations of the shake table are summarized in the test equipment chapter. There were 966 simulation records to choose from. Considering the peak limitations of displacement (D), velocity (V), acceleration (A), and oil flow (Q) individually, the number of records meeting the criteria was 282, 732, 966, and 700 respectively. When considering the combined limits of D+V+A, the suitable records was 271 and considering D+V+A+Q, the number of suitable records was 251. Additionally, for comparison with analytical studies by Wood et al. (2009) it was desirable to select records that had similar crack cycle counting statistics as the cracking protocol (CP1) of Wood et al. (2009). When considering all limitations, and searching for
records with cycle counts compatible with CP1, the number of suitable records was reduced to 31.

<table>
<thead>
<tr>
<th>Description of Limit</th>
<th>Table Maximum Allowable</th>
</tr>
</thead>
<tbody>
<tr>
<td>Peak Displacement</td>
<td>± 5.5 in</td>
</tr>
<tr>
<td>Peak Velocity</td>
<td>± 36.0 in/s</td>
</tr>
<tr>
<td>Peak Acceleration</td>
<td>± 3 g</td>
</tr>
<tr>
<td>Peak Oil Flow</td>
<td>420 gpm</td>
</tr>
</tbody>
</table>

**Table 4.8 Shake table limitations**

4.7.3 Correlation Test Motions

The numerical simulations of Wood et al. (2009) considered 21 input ground motions and five different building heights. The details of the ground motions are summarized in Table 4.9. From these, 20 floor motion (FM01 to FM20) records were selected that used eight of the ground motions simulated in all five buildings. The selected floor motions FM01 to FM20 are listed in Table 4.10. The peak displacement, velocity, and acceleration are all absolute quantities of the floor given at 100% scale. The resulting floor acceleration histories had to then be scaled before being used as input to the shake table tests in order to achieve the desired level of force in the anchors. The scale factors were selected to achieve peak anchor forces between 10 and 50% of $N_{um.cr}$, the mean tension strength in cracked concrete with a static crack width of 0.8 mm. Table 4.10 summarizes the resulting scale factors used for the flexible and stiff WALLE correlation tests.
Table 4.9 Selected ground motion details

<table>
<thead>
<tr>
<th>GM No.</th>
<th>Event Folder Name</th>
<th>Event Name</th>
<th>Used?</th>
<th>Floor Motion No.</th>
<th>Date</th>
<th>Location</th>
<th>Focal Mechanism</th>
<th>Magnitude (Mw)</th>
<th>Distance (km)</th>
<th>Scale Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Baja California</td>
<td>baja</td>
<td>Y</td>
<td>FM11, 15</td>
<td>Juli 2, 1987</td>
<td>Mexicali, Mexico</td>
<td>Strike-Slip</td>
<td>5.50</td>
<td>3.70</td>
<td>0.78</td>
</tr>
<tr>
<td>2</td>
<td>Cape Mendocino</td>
<td>capemend</td>
<td>N</td>
<td>--</td>
<td>Apr 25, 1992</td>
<td>Cape Mendocino, CA, USA</td>
<td>Strike-Slip</td>
<td>7.01</td>
<td>14.53</td>
<td>1.36</td>
</tr>
<tr>
<td>3</td>
<td>Cape Mendocino</td>
<td>capemend</td>
<td>N</td>
<td>--</td>
<td>Apr 25, 1992</td>
<td>Cape Mendocino, CA, USA</td>
<td>Strike-Slip</td>
<td>7.01</td>
<td>10.36</td>
<td>1.45</td>
</tr>
<tr>
<td>4</td>
<td>Chi Chi</td>
<td>chichi3</td>
<td>N</td>
<td>--</td>
<td>Sep 25, 1999</td>
<td>Taichung City, Taiwan</td>
<td>Reverse</td>
<td>6.30</td>
<td>11.52</td>
<td>1.89</td>
</tr>
<tr>
<td>5</td>
<td>Friuli</td>
<td>friuli</td>
<td>Y</td>
<td>FM07, 10</td>
<td>Nov 12, 1999</td>
<td>Friuli, Italy</td>
<td>Reverse</td>
<td>6.50</td>
<td>14.97</td>
<td>1.82</td>
</tr>
<tr>
<td>6</td>
<td>Gazli</td>
<td>gazi</td>
<td>N</td>
<td>--</td>
<td>May 17, 1976</td>
<td>Gazli, USSR</td>
<td>Reverse</td>
<td>6.80</td>
<td>12.82</td>
<td>1.03</td>
</tr>
<tr>
<td>7</td>
<td>Irpina</td>
<td>irpina</td>
<td>Y</td>
<td>FM01, 09</td>
<td>Nov 23, 1980</td>
<td>Ispina, Italy</td>
<td>Normal</td>
<td>6.90</td>
<td>15.04</td>
<td>2.68</td>
</tr>
<tr>
<td>8</td>
<td>Kobe</td>
<td>kobe</td>
<td>N</td>
<td>--</td>
<td>Jan 16, 1995</td>
<td>Kobe, Japan</td>
<td>Strike-Slip</td>
<td>6.90</td>
<td>7.08</td>
<td>1.17</td>
</tr>
<tr>
<td>9</td>
<td>Kobe</td>
<td>kobe00</td>
<td>Y</td>
<td>FM02, 12, 16, 18</td>
<td>Jan 16, 1995</td>
<td>Nishi-Akashi, Japan</td>
<td>Strike-Slip</td>
<td>6.90</td>
<td>8.70</td>
<td>0.91</td>
</tr>
<tr>
<td>10</td>
<td>Landers</td>
<td>land</td>
<td>N</td>
<td>--</td>
<td>Jun 28, 1992</td>
<td>Lucerne, CA, USA</td>
<td>Strike-Slip</td>
<td>7.28</td>
<td>10.37</td>
<td>1.82</td>
</tr>
<tr>
<td>11</td>
<td>Loma Prieta</td>
<td>loma</td>
<td>N</td>
<td>--</td>
<td>Oct 18, 1989</td>
<td>San Jose, CA, USA</td>
<td>Rev-Oblique</td>
<td>6.93</td>
<td>14.69</td>
<td>2.06</td>
</tr>
<tr>
<td>12</td>
<td>Loma Prieta</td>
<td>lomap</td>
<td>N</td>
<td>--</td>
<td>Oct 18, 1989</td>
<td>Sanatoga, CA, USA</td>
<td>Rev-Oblique</td>
<td>6.93</td>
<td>9.31</td>
<td>1.23</td>
</tr>
<tr>
<td>14</td>
<td>Nahanni</td>
<td>naha</td>
<td>N</td>
<td>--</td>
<td>Dec 23, 1985</td>
<td>Nahanni, Canada</td>
<td>Reverse</td>
<td>6.76</td>
<td>6.52</td>
<td>2.37</td>
</tr>
<tr>
<td>15</td>
<td>Northridge</td>
<td>ngra</td>
<td>Y</td>
<td>FM06, 17</td>
<td>Jan 17, 1994</td>
<td>Castaic, CA, USA</td>
<td>Reverse</td>
<td>6.69</td>
<td>20.72</td>
<td>0.97</td>
</tr>
<tr>
<td>16</td>
<td>Northridge</td>
<td>north</td>
<td>Y</td>
<td>FM03, 08</td>
<td>Jan 17, 1994</td>
<td>Los Angeles, CA, USA</td>
<td>Reverse</td>
<td>6.69</td>
<td>22.49</td>
<td>1.57</td>
</tr>
<tr>
<td>17</td>
<td>San Fernando</td>
<td>sanf</td>
<td>N</td>
<td>--</td>
<td>Feb 9, 1971</td>
<td>Castaic, CA, USA</td>
<td>Reverse</td>
<td>6.61</td>
<td>25.36</td>
<td>2.27</td>
</tr>
<tr>
<td>18</td>
<td>San Salvador</td>
<td>salv</td>
<td>Y</td>
<td>FM04</td>
<td>Oct 10, 1986</td>
<td>San Salvador, El Salvador</td>
<td>Strike-Slip</td>
<td>5.80</td>
<td>9.95</td>
<td>1.16</td>
</tr>
<tr>
<td>19</td>
<td>Superstition</td>
<td>super</td>
<td>Y</td>
<td>FM05, 13, 14, 19, 20</td>
<td>Nov 24, 1987</td>
<td>Superstition Mtn, CA, USA</td>
<td>Strike-Slip</td>
<td>6.54</td>
<td>7.50</td>
<td>0.87</td>
</tr>
<tr>
<td>20</td>
<td>Tabas</td>
<td>tabas</td>
<td>N</td>
<td>--</td>
<td>Jun 28, 1991</td>
<td>Tabas, Iran</td>
<td>Reverse</td>
<td>7.35</td>
<td>2.05</td>
<td>0.55</td>
</tr>
<tr>
<td>21</td>
<td>Victoria</td>
<td>victo</td>
<td>N</td>
<td>--</td>
<td>Jun 9, 1980</td>
<td>Mexicali, Mexico</td>
<td>Strike-Slip</td>
<td>6.33</td>
<td>14.37</td>
<td>1.19</td>
</tr>
<tr>
<td>22</td>
<td>TOTAL USED</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>8</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Figure 4.15 and Figure 4.16 display the experimentally achieved acceleration time histories at the slab, which served as input to WALLE during the flexible WALLE variable phase correlation tests. Figure 4.17 and Figure 4.18 display the achieved acceleration time histories at the slab which served as input to WALLE during the stiff WALLE variable phase correlation tests. The acceleration histories display a broad range of motion characteristics in terms of amplitude, effective duration, number of cycles, and frequency content.
Table 4.10 Selected correlation test input motion records

<table>
<thead>
<tr>
<th>No.</th>
<th>Building</th>
<th>Floor</th>
<th>Earthquake</th>
<th>Peak Disp. 100% scale (in)</th>
<th>Peak Velocity 100% scale (in/s)</th>
<th>Peak Accel. 100% scale (g)</th>
<th>Peak Oil Flow 100% scale (gpm)</th>
<th>Scale Factor (STIFF WALLE)</th>
<th>Scale Factor (FLEXIBLE WALLE)</th>
</tr>
</thead>
<tbody>
<tr>
<td>FM01</td>
<td>2st</td>
<td>1</td>
<td>irpina</td>
<td>5.21</td>
<td>18.38</td>
<td>0.63</td>
<td>355</td>
<td>24%</td>
<td>8%</td>
</tr>
<tr>
<td>FM02</td>
<td>2st</td>
<td>1</td>
<td>kobe00</td>
<td>3.67</td>
<td>18.05</td>
<td>0.95</td>
<td>359</td>
<td>36%</td>
<td>8%</td>
</tr>
<tr>
<td>FM03</td>
<td>2st</td>
<td>1</td>
<td>north</td>
<td>4.63</td>
<td>24.40</td>
<td>1.32</td>
<td>420</td>
<td>23%</td>
<td>6%</td>
</tr>
<tr>
<td>FM04</td>
<td>2st</td>
<td>1</td>
<td>sanf</td>
<td>4.16</td>
<td>25.37</td>
<td>0.78</td>
<td>347</td>
<td>32%</td>
<td>8%</td>
</tr>
<tr>
<td>FM05</td>
<td>4st</td>
<td>1</td>
<td>super</td>
<td>2.46</td>
<td>16.70</td>
<td>0.59</td>
<td>267</td>
<td>30%</td>
<td>15%</td>
</tr>
<tr>
<td>FM06</td>
<td>4st</td>
<td>1</td>
<td>nga</td>
<td>3.65</td>
<td>19.03</td>
<td>0.61</td>
<td>311</td>
<td>60%</td>
<td>23%</td>
</tr>
<tr>
<td>FM07</td>
<td>4st</td>
<td>2</td>
<td>friuli</td>
<td>2.96</td>
<td>24.06</td>
<td>0.79</td>
<td>353</td>
<td>60%</td>
<td>15%</td>
</tr>
<tr>
<td>FM08</td>
<td>4st</td>
<td>2</td>
<td>north</td>
<td>5.14</td>
<td>19.49</td>
<td>0.79</td>
<td>325</td>
<td>70%</td>
<td>35%</td>
</tr>
<tr>
<td>FM09</td>
<td>8st</td>
<td>1</td>
<td>irpina</td>
<td>5.11</td>
<td>15.93</td>
<td>0.40</td>
<td>224</td>
<td>45%</td>
<td>15%</td>
</tr>
<tr>
<td>FM10</td>
<td>8st</td>
<td>3</td>
<td>friuli</td>
<td>4.15</td>
<td>21.07</td>
<td>0.59</td>
<td>273</td>
<td>30%</td>
<td>10%</td>
</tr>
<tr>
<td>FM11</td>
<td>8st</td>
<td>5</td>
<td>baja</td>
<td>4.60</td>
<td>25.67</td>
<td>0.65</td>
<td>336</td>
<td>27%</td>
<td>18%</td>
</tr>
<tr>
<td>FM12</td>
<td>8st</td>
<td>5</td>
<td>kobe00</td>
<td>5.08</td>
<td>22.04</td>
<td>0.45</td>
<td>323</td>
<td>72%</td>
<td>19%</td>
</tr>
<tr>
<td>FM13</td>
<td>12st</td>
<td>2</td>
<td>super</td>
<td>1.89</td>
<td>14.21</td>
<td>0.56</td>
<td>214</td>
<td>28%</td>
<td>10%</td>
</tr>
<tr>
<td>FM14</td>
<td>12st</td>
<td>5</td>
<td>super</td>
<td>3.05</td>
<td>21.83</td>
<td>0.60</td>
<td>279</td>
<td>87%</td>
<td>23%</td>
</tr>
<tr>
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<td>12st</td>
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<td>baja</td>
<td>4.46</td>
<td>28.04</td>
<td>0.48</td>
<td>286</td>
<td>75%</td>
<td>25%</td>
</tr>
<tr>
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<td>12st</td>
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<td>kobe00</td>
<td>5.14</td>
<td>19.45</td>
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<td>300</td>
<td>120%</td>
<td>40%</td>
</tr>
<tr>
<td>FM17</td>
<td>20st</td>
<td>4</td>
<td>nga</td>
<td>3.92</td>
<td>20.51</td>
<td>0.46</td>
<td>206</td>
<td>60%</td>
<td>20%</td>
</tr>
<tr>
<td>FM18</td>
<td>20st</td>
<td>4</td>
<td>kobe00</td>
<td>4.22</td>
<td>15.49</td>
<td>0.45</td>
<td>179</td>
<td>90%</td>
<td>20%</td>
</tr>
<tr>
<td>FM19</td>
<td>20st</td>
<td>9</td>
<td>super</td>
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<td>19.98</td>
<td>0.46</td>
<td>293</td>
<td>75%</td>
<td>25%</td>
</tr>
<tr>
<td>FM20</td>
<td>20st</td>
<td>18</td>
<td>super</td>
<td>5.32</td>
<td>23.75</td>
<td>0.47</td>
<td>295</td>
<td>36%</td>
<td>81%</td>
</tr>
</tbody>
</table>
Figure 4.15 Experimentally achieved variable phase correlation input motions FM01 to FM10 (V2 Flexible WALLE) PIA=peak input acceleration.
Figure 4.16 Experimentally achieved variable phase correlation input motions FM11 to FM20 (V2 Flexible WALLE) PIA=peak input acceleration.
Figure 4.17 Experimentally achieved variable phase correlation input motions FM01 to FM11 (V2 Rigid WALLE) PIA=peak input acceleration.
Figure 4.18 Experimentally achieved variable phase correlation input motions FM11 to FM20 (V2 Rigid WALLE) PIA=peak input acceleration.
Using the target scale factors from Table 4.10, Figure 4.19 and Figure 4.20 compare the achieved acceleration spectra with the target spectra for the flexible and stiff WALLE correlation tests, respectively. Overshooting of the spectra accelerations in the short period range is evident due to stick slip behavior of the table; however, in the period range of interest for the flexible WALLE ($T=0.25$ seconds) the achieved spectra matches the target spectra more closely.

A note of interest is that while two sets of correlation motion were conducted (Version 1 and Version 2), an internal data acquisition filter was inadvertently applied to the anchor load channels only during the Version 1 tests. The filter was at a higher cutoff frequency than the anchor loading and is not expected to have a significant effect on the results; however, as a precaution, the first series of correlation tests were repeated. The repeated tests are denoted in version 2. Moreover, due to the close repeatability of the motions from Version 1 to Version 2 it is convenient to present results from the Version 2 tests only from this point forward.
Figure 4.19 Two percent damped elastic acceleration response spectra for the variable phase correlation tests: target, version 1, and version 2 (flexible WALLE), target scale factor is noted
Figure 4.20 Two percent damped elastic acceleration response spectra for the variable phase correlation tests: target, version 1, and version 2 (rigid WALLE), target scale factor is noted.
4.7.4 Failure Test Motion

Motion FM02 was selected from the suite of correlation test floor motions for use in the failure tests. The reference motion (FM02) is the 1st floor level response from a 2-story frame-braced building subjected to the Kobe00 motion (Wood et al., 2009). It was selected as a reference failure motion mainly because of its large spectral acceleration at the period of flexible WALLE, approximately 0.25 seconds. This motion would guarantee that large anchor tension loads (up to failure) could be imposed without exhausting the limits of the shake table. Figure 4.21 shows the target versus achieved FM02 motion, where the achieve and motions are the slab acceleration, slab displacement and average crack width for test WALLE.F.N7.CC08.FM02.100P (Test No. 141). Figure 4.22 shows the same tests windowed for clarity. It is noted that the acceleration generally matches the target except at high frequencies, where the achieved is larger than the target. There is a discrepancy between the target and achieved displacement, but this is because the target displacement is the target acceleration double integrated to displacement but is not baseline corrected. The SDSU uncracked tests were conducted in displacement control using the non baseline corrected version of the displacement, therefore the target displacement was left as uncorrected. The achieved average crack width matches closely the target crack width. Therefore it is concluded that the CCILR and shake table adequately reproduce the failure motion for the purposes of this testing program.
Figure 4.21 Target versus achieved time histories for failure test
WALLE.F.N7.CC08.FM02.100P ($w$ is the average crack width at the four anchors)

Figure 4.22 Target versus achieved time histories for failure test
WALLE.F.N7.CC08.FM02.100P windowed from 6 to 12 seconds ($w$ is the average crack width at the four anchors)
Another way to assess the robustness of the shake table in achieving the failure test motions is to examine the record in the spectral domain. Specifically one may use the acceleration spectrum, as most floor mounted mechanical and electrical components may be classified as acceleration sensitive (CSA, 2001). The International Code Council (ICC) publishes an acceptance criteria for seismic certification of nonstructural components by shake-table testing (AC156, 2010). The AC156 standard uses a broad band random input motion with a total duration of 30 seconds with 20 seconds minimum of strong motion. A comparison of the failure motion (FM02) at 100% scale with the AC156 Required Response Spectrum (RRS) reveals that the horizontal response spectra is comparable to the design scenario for the building location of Charter Oaks, California (i.e. $S_{DS}=1.34\, \text{g}$, and mounted on the first floor of a two story building, $z/h=0.5$) (Figure 4.23).

![Figure 4.23 Comparison of reference failure motion, 100% FM02, acceleration response spectra calculated at 5% damping, with ICC-AC156 test standard](image-url)
The reference floor motion spectrum is less than AC156 in the 1.33 to 2 Hz and 5.5-7 Hz range, and greater in the 3 to 5.5 Hz range and above 7 Hz. It is noted that AC156 response spectra are calculated at 5% damping. In terms of input motion duration; however, the FM02 motion has a significantly shorter effective duration of 10 seconds (FM02, 100%), as calculated by the duration between the time at 5% and 95% Arias intensity, (Arias, 1970), 13 seconds versus 20 seconds required by AC156, causing FM02 to be less damaging overall due to its shorter duration. The AC156 spectrum for this design NCS has a peak 5% damped spectral acceleration of 2.14 g which corresponds to the ASCE 7, 2005 code prescribed component force of $F_p/W_p = 2.14$ g. An example calculation of the code component force is given in Figure 4.24. The flexible WALLE has a natural frequency of 4 Hz therefore the amplification factor $a_p$ was taken as 2.5 and the WALLE was designed to remain elastic therefore $R_p$ was taken as 1.0. This calculation also draws attention to the fact that the anchors were intentionally sized to have failure at the specified failure motion. In this design, the code calculated demand force is approximately 3.5 times the anchor capacity, considering combined shear and tension, and 2.1 times the anchor capacity, considering tension, for a $\frac{1}{2}''$ diameter expansion anchor located in cracked concrete.

To assess the fidelity of the shake table-CCILR system it is useful to compare the achieved acceleration spectra to the target acceleration spectra. All test spectra are calculated for a SDOF with 2% critical damping. This level was representative of the WALLE at the beginning of each test prior to significant damage to the anchors. Figure 4.25 to 5.16 display the acceleration spectra target versus achieved for the last failure test for each anchor sequence for Epoxy, Expansion ($w_{max} = 0.5$mm), Expansion ($w_{max} = 0.8$mm) and Drop-In undercut anchors respectively. Again, except for the high frequency overshoot due to stick-slip behavior of the table the FM02 failure motions are considered to be adequately reproduced for the purposes of this study.
Figure 4.24 Example calculation of nonstructural component forces per ASCE 7, 2005

for an NCS with $S_D = 1.34$ g and a component elevation factor $z/h = 0.5$
Figure 4.25 Target versus achieved two percent damped elastic spectral acceleration for failure test WALLE.F.N4.CC08.FM02.30P

Figure 4.26 Target versus achieved two percent damped elastic spectral acceleration for failure test V2.WALLE.F.N5.CC05.FM02.100P
Figure 4.27 Target versus achieved two percent damped elastic spectral acceleration for failure test WALLE.F.N5.CC08.FM02.125P

Figure 4.28 Target versus achieved two percent damped elastic spectral acceleration for failure test WALLE.F.N7.CC08.FM02.100P
4.7.5 Repeatability of Failure Motions

During the failure tests, the input floor accelerations were amplitude scaled and imposed at increasing scale factors until failure of the anchors occurred. Figure 4.30 summarizes the acceleration spectra of the input motions as measured at the slab at the base of WALLE. To compare tests conducted at different amplitude scales, Figure 4.31 summarizes the acceleration spectra of the input motions as measured at the slab at the base of WALLE rescaled for comparison to the original 100% target motion, as follows:

\[ S_{a_{\text{normalized}}} = S_{a_{\text{achieved}}} \left( \frac{100\%}{\text{Target Scale}} \right) \]

The stick-slip characteristic of the shake table bearings is evident in the short period overshoot of the target spectra; however, the measured motions match the target motion well in the period range of interest for WALLE between 0.2 to 1.0 seconds. The spectral
acceleration at the period of WALLE range from 2.5 to over 10g during the failure tests.

Figure 4.32 summarizes the displacement spectra of the input motions as measured at the slab at the base of WALLE. To compare tests conducted at different amplitude scales, Figure 4.33 summarizes the displacement spectra of the input motions all scaled to 100%. The spectral displacement at the period of WALLE range from 0.2” to 0.9” during the failure tests.

Figure 4.30 Two percent damped elastic spectral acceleration of all failure tests
Figure 4.31 Two percent damped elastic spectral acceleration of all failure tests rescaled to 100 percent scale

Figure 4.32 Two percent damped elastic spectral displacement of all failure tests
4.7.6 Achieved Scale Factors

Because the shake table and CCILR test apparatus are a non-linear system, the achieved motions at the slab at the base of WALLE that serve as input to the WALLE differ in dynamic characteristics from the target motions. As described earlier, the motions at the base of WALLE are different from the target motions mainly due to two factors: 1) the stick-slip dynamic characteristics of the shake table, and 2) the additional accelerations induces by the actuators that control the dynamic cracking of the slab and resulting movement of the slab in the direction of motion, which can range up to 0.5” (12 mm) of peak displacement due to movement of the slab from cracking alone. To determine the achieved scale of a record, the spectra was evaluated only over a specific range of periods that were felt to be representative of WALLE’s response, i.e., its natural period including an allowance for period elongation during the test. The period range of interest is defined as the period range of the minimum and
maximum WALLE period for the test group under consideration. WALLE period was determined by calculating a Frequency Response Function (FRF). This technique is described in more detail in the section of system identification. The test groups considered are as follows: 1) Failure tests, 2) Variable phase correlation (stiff WALLE), and 3) Variable phase correlation tests (flexible WALLE). Table 4.11 summarizes WALLE period statistics for the test groups under consideration including; the minimum, maximum, average, and plus minus one standard deviation. The period range used in the analysis of the spectra is highlighted.

Table 4.12 summarizes the target scale factors used during the failure tests as well as the achieved scale factors which are defined by the equation below as the average ratio of the spectral acceleration recorded as input to the WALLE to that of the target floor motion in the period range of interest where $i$ to $n$ are the periods within the period range.

$$Actual \ Scale = \frac{1}{n} \sum_{i=1}^{n} \frac{Sa_{\text{achieved}}(T_i)}{Sa_{\text{target}}(T_i)}$$

As can be seen in Table 4.12, the achieved scale factor overshoots by approximately 20% for low amplitude scaled motions with scale factors from 30-100%. Fidelity increases with higher motion amplitudes. For reference, the 2% damped spectral acceleration at $T_{\text{WALLE}}=0.25$ sec is reported along with the WALLE period during the failure motion. WALLE experiences significant period elongation in the higher amplitude motion tests. This is generally due to permanent displacement of the anchors and a shift in WALLE from anchored behavior to “unanchored” rocking-type behavior. Table 4.13 repeats this data for all failure tests sequences with increasing scale up to anchor failure. Table 4.14 presents the actual scale achieved during the variable phase correlation testing for Version 1 and Version 2 tests. There is general agreement in repeatability of scale from the Version 1 to Version2 tests, generally the scales between two versions of the tests were within 5% of each other. One trend that is evident is the large overshoot in scale for the stiff WALLE correlation tests, this is
because the WALLE has a low period, 0.1 sec and the shake table tends to overshoot in the short period motion range due to stick-slip behavior of the table. Table 4.15 summarizes the acceleration levels during the failure tests. Peak input acceleration at the slab ranged from 0.5-2.0g while spectral accelerations at the WALLE period ranged from 2.7-11.0g at anchor failure.

Table 4.11 WALLE period range of interest

<table>
<thead>
<tr>
<th>Period Parameter</th>
<th>All Failure Tests</th>
<th>Correlation Tests (stiff WALLE)</th>
<th>Correlation Tests (flexible WALLE)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average period [s]</td>
<td>0.33</td>
<td>0.12</td>
<td>0.26</td>
</tr>
<tr>
<td>Minimum period [s]</td>
<td>0.16</td>
<td>0.10</td>
<td>0.23</td>
</tr>
<tr>
<td>Maximum period [s]</td>
<td>1.12</td>
<td>0.14</td>
<td>0.30</td>
</tr>
<tr>
<td>µ + σ [s]</td>
<td>0.52</td>
<td>0.13</td>
<td>0.28</td>
</tr>
<tr>
<td>µ - σ [s]</td>
<td>0.14</td>
<td>0.11</td>
<td>0.25</td>
</tr>
<tr>
<td>Period range used [s]</td>
<td>0.20 to 0.75</td>
<td>0.10 to 0.14</td>
<td>0.22 to 0.30</td>
</tr>
</tbody>
</table>

Table 4.12 Re-evaluated scale factors for final failure tests

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Test Name</th>
<th>Target Scale Factor (%)</th>
<th>Achieved Scale Factor* (%)</th>
<th>Sa @ T=0.25 s, ξ=2% (g)</th>
<th>WALLE Period T_WALLE (s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>182</td>
<td>WALLE_F_N4_UC_30_1</td>
<td>30</td>
<td>41</td>
<td>2.69</td>
<td>0.21</td>
</tr>
<tr>
<td>132</td>
<td>WALLE_F_N4_CC08_FM02_30P</td>
<td>30</td>
<td>37</td>
<td>2.76</td>
<td>0.56</td>
</tr>
<tr>
<td>185</td>
<td>WALLE_F_N5_UC_150_1</td>
<td>150</td>
<td>152</td>
<td>10.83</td>
<td>0.16</td>
</tr>
<tr>
<td>146</td>
<td>V2_WALLE_F_N5_CC05_FM02_100P</td>
<td>100</td>
<td>120</td>
<td>8.79</td>
<td>1.05</td>
</tr>
<tr>
<td>57</td>
<td>WALLE_F_N5_CC08_FM02_125P</td>
<td>125</td>
<td>148</td>
<td>11.05</td>
<td>0.21</td>
</tr>
<tr>
<td>141</td>
<td>WALLE_F_N7_CC08_FM02_100P</td>
<td>100</td>
<td>120</td>
<td>8.92</td>
<td>1.12</td>
</tr>
<tr>
<td>51</td>
<td>WALLE_F_N9_CC08_FM02_60P</td>
<td>60</td>
<td>71</td>
<td>5.00</td>
<td>0.59</td>
</tr>
<tr>
<td>120</td>
<td>V2_WALLE_F_N9_CC08_FM02_60P</td>
<td>60</td>
<td>72</td>
<td>5.21</td>
<td>0.34</td>
</tr>
</tbody>
</table>

*The average ratio of spectral acceleration of the recorded slab motion to that of the target floor motion in the period range of interest defined by the minimum and maximum fundamental period of WALLE during that test group.
Table 4.13 Re-evaluated scale factors for failure test sequences

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Test Name</th>
<th>Target Scale Factor (%)</th>
<th>Actual Scale Factor * (%)</th>
<th>Sa @ T=0.25 s ξ=2%</th>
<th>WALLE Period T_{WALLE} (s)</th>
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</tr>
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<td>27</td>
<td>1.73</td>
<td>0.21</td>
</tr>
<tr>
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<td>WALLE_F_N4_UC_30_1</td>
<td>30</td>
<td>41</td>
<td>2.69</td>
<td>0.21</td>
</tr>
<tr>
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<td>13</td>
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<td>0.26</td>
</tr>
<tr>
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<td>0.27</td>
</tr>
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<td>37</td>
<td>2.76</td>
<td>0.56</td>
</tr>
<tr>
<td>183</td>
<td>WALLE_F_N5_UC_20_1</td>
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<td>1.62</td>
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<tr>
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<td>WALLE_F_N5_UC_100_1</td>
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<td>6.74</td>
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<td>WALLE_F_N5_UC_150_1</td>
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<td>152</td>
<td>10.83</td>
<td>0.16</td>
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<td>0.28</td>
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<td>8.79</td>
<td>1.05</td>
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<td>38</td>
<td>2.82</td>
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<tr>
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<td>77</td>
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<td>0.36</td>
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<tr>
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<td>WALLE_F_N5_CC08_FM02_125P</td>
<td>125</td>
<td>148</td>
<td>11.05</td>
<td>0.21</td>
</tr>
<tr>
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<td>11</td>
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<td>0.29</td>
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<tr>
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<td>0.34</td>
</tr>
<tr>
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<td>24</td>
<td>1.55</td>
<td>0.37</td>
</tr>
<tr>
<td>138</td>
<td>WALLE_F_N7_CC08_FM02_25P</td>
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<td>29</td>
<td>1.94</td>
<td>0.38</td>
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<tr>
<td>139</td>
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<td>35</td>
<td>41</td>
<td>2.78</td>
<td>0.36</td>
</tr>
<tr>
<td>140</td>
<td>WALLE_F_N7_CC08_FM02_60P</td>
<td>60</td>
<td>72</td>
<td>5.14</td>
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<tr>
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<td>WALLE_F_N7_CC08_FM02_100P</td>
<td>100</td>
<td>120</td>
<td>8.92</td>
<td>1.12</td>
</tr>
<tr>
<td>48</td>
<td>WALLE_F_N9_CC08_FM02_15P</td>
<td>15</td>
<td>21</td>
<td>1.60</td>
<td>0.27</td>
</tr>
<tr>
<td>49</td>
<td>WALLE_F_N9_CC08_FM02_20P</td>
<td>20</td>
<td>25</td>
<td>1.69</td>
<td>0.27</td>
</tr>
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<td>50</td>
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<td>48</td>
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<td>71</td>
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</tr>
<tr>
<td>118</td>
<td>V2_WALLE_F_N9_CC08_FM02_20P</td>
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<td>26</td>
<td>2.01</td>
<td>0.30</td>
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<td>119</td>
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<td>48</td>
<td>3.34</td>
<td>0.34</td>
</tr>
<tr>
<td>120</td>
<td>V2_WALLE_F_N9_CC08_FM02_60P</td>
<td>60</td>
<td>72</td>
<td>5.21</td>
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*The average ratio of spectral acceleration of the recorded slab motion to that of the target floor motion in the period range of interest defined by the minimum and maximum fundamental period of WALLE during that test group.
<table>
<thead>
<tr>
<th>Test Name</th>
<th>Target Scale Factor (%)</th>
<th>V1 Actual Scale Factor (%)</th>
<th>V2 Actual Scale Factor (%)</th>
<th>Difference in Scale (V2-V1)/V1 (%)</th>
</tr>
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<tr>
<td><strong>Stiff WALLE correlation tests ($T_{WALLE} \sim 0.10 \text{ s.}$)</strong></td>
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<td>81</td>
<td>119</td>
<td>117</td>
<td>-1.7</td>
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Table 4.15 Accelerations for final failure tests

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Test Name</th>
<th>Achieved Scale Factor* (%)</th>
<th>Sa @ T=0.25 s ξ=2% (g)</th>
<th>Peak Input Accel. PIA (g)</th>
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<tr>
<td>182</td>
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<td>185</td>
<td>WALLE_F_N5_UC_150_1</td>
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<td>10.83</td>
<td>1.682</td>
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<td>146</td>
<td>V2_WALLE_F_N5_CC05_FM02_100P</td>
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<td>57</td>
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<td>72</td>
<td>5.21</td>
<td>1.032</td>
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</table>

Using the re-evaluated “actual” scale factors, Figure 4.34 and Figure 4.35 compare the achieved acceleration spectra with the target spectra for the flexible and stiff WALLE correlation tests respectively. Overshoot of the spectra in the short period range is evident; however, in the period range of interest for the flexible WALLE (T=0.25 seconds) the spectra matches well. It is concluded that the correlation motions generally match the acceleration spectra in the period range of interest for the flexible WALLE and that the tests were adequately repeatable from Version 1 to Version 2 tests.
Figure 4.34 Acceleration spectra for the variable phase correlation tests: target, version 1, and version 2 (flexible WALLE), showing re-evaluated scale factor.
Figure 4.35 Acceleration spectra for the variable phase correlation tests: target, version 1, and version 2 (rigid WALLE), showing re-evaluated scale factor.
4.7.7 Achieved Crack Width

Another measure of the fidelity of the input motions is in terms of the achieved versus target crack width. Watkins et al. (2011) describe in detail the fidelity of the CCILR system in producing cyclic crack histories and conclude that the CCILR is able to accurately reproduce dynamic cyclic crack widths up to 1.3 mm with an accuracy of +/- 0.1 mm. It is noted that 0.1 mm is the average thickness of a typical human hair. Figure 4.36 summarizes the maximum crack width achieved during the variable phase correlation tests showing that 97% of the maximum crack width measurements were within the scatter band of 0.7 to 0.9 mm for a target maximum width of 0.8 mm. Figure 4.37 summarizes the maximum crack width achieved during the failure test series. The failure tests still generally fall within +/- 0.1 mm (68% of maximum crack width measurements) and +/- 0.2 mm (90% of maximum crack width measurements) however, the results display more scatter in the maximum crack width, particularly for the lower target crack width of 0.5 mm. A few tests with $w_{max}$, target = 0.8 mm, resulted in maximum crack widths that were +0.2 mm larger than targeted. An inspection of the anchor load and crack time histories reveals that this occurs when an anchor is highly loaded in tension and the expansion forces of the anchor drive the crack open further than targeted (e.g. large anchor tension loads, $N$, corresponding with overshoot of $w_{max}$). Additionally, the overshoot of always occur on the South pair of anchors for motion FM02, this is because the asymmetric nature of that floor motion which causes maximum anchor tension loads on the South pair of anchors.
Figure 4.36 Maximum achieved crack width, $w_{\text{max}}$, at each anchor: correlation tests

Figure 4.37 Maximum achieved crack width, $w_{\text{max}}$, at each anchor: failure test series
4.7.8 Summary Remarks on Motions Used

Based on the input motion selection process and review of the achieved motions, the following observations can be made:

- A suite of 20 floor input motions (FM01-FM20) was selected from the Wood et al. (2009) nonlinear building simulation results thus providing a random sampling of motions with a wide variety of characteristics in terms of magnitude, earthquake, distance, frequency content, and effective duration.

- The FM01-FM20 records consisted of an absolute floor acceleration history and the corresponding crack width history of a beam in flexure at that floor. The records were amplitude scaled to achieved the desired level of anchor force, 10 to 50% $N_{um,cr}$ for the correlation tests, where $N_{um,cr}$ is the mean ultimate strength from the reference tests for the anchor located in static cracks having a crack width of either 0.8 mm or 0.5 mm. Curvature histories were converted to crack histories and scaled such that $w_{max} = 0.5$ or 0.8 mm (depending on that test target).

- The FM02 motion was selected as the reference floor motion that was used during all failure tests. The motion was amplitude scaled until failure of the anchor type under consideration was achieved. Scale factors at failure ranged from 30 to 125 percent.

- The achieved motions generally matched the target motions with the exception that the achieved motions had more high frequency content above 4Hz (i.e. overshot the target spectra at periods below 0.25 seconds). This high frequency overshoot was more evident at the low amplitude levels used in the variable phase correlation tests. There was less effect during the failure tests, which all used the flexible WALLE ($T_{WALLE} = 0.25$ seconds). As WALLE’s
period is expected to elongate during shaking, the impact of the high frequency acceleration overshoot is anticipated to be lessened. At periods greater than 0.25 seconds, the achieved motions matched quite well with the target.

- The FM02 failure motion was reasonably repeatable throughout the failure test series, allowing for a meaningful comparison of anchor behavior for the various anchor types tested.

- The FM02 floor motion acceleration spectra at 100% amplitude scale envelopes the industry standard AC156 spectra for the design earthquake floor acceleration levels for the design site of Charter Oaks, CA for a nonstructural component located on the second floor of a two story building ($S_{DS} = 1.34g$, $z/h=0.5$). The FM02 acceleration spectra is 2.5 times larger than the required response spectrum around a frequency of 4 Hz, which is approximately the natural frequency of the building and the natural frequency of flexible WALLE.

- The achieved crack width histories are considered to have reasonably satisfied the target crack width histories for the purpose of this testing program.

### 4.8 Nomenclature

This section describes the nomenclature used for crack width, anchor locations, and anchor load. For each parameter, the subscript “$i$” is used to denote values at discrete time instances at which the data is sampled. The subscript “$j$” is used to denote different anchors or different data channels. For example anchor load ($N$) at time “$i$” for anchor “$j$” would be named $N_{i,j}$. The crack width measured and studied ($w$) is the sum of crack opening ($\Delta w$) and the hairline crack ($w_{hairline}$) i.e.:
Because the hairline crack width is typically very small, on the order of 0.05mm for reinforced concrete members detailed in accordance with code procedures, often the crack width \( w \) and change in crack width \( \Delta w \) are used interchangeably. Before the start of each test, the cracks were actively closed by applying a pre-compression force of 100 kips (50 kips per actuator) or approximately \( 5\% A_g f_c' \) where \( A_g = \) gross sectional area of the section (= 420 in\(^2\)) and \( f_c' = \) unconfined compression (4 ksi). This serves to minimize the hairline crack width. For this reason, the plots in this report all reference \( w \), however, this is actually the measured change in crack width \( \Delta w \).

To study the magnitude of anchor axial tension load \( N \) at a particular crack width \( w \) for a variety of different tests conducted at differing seismic input levels and hence anchor load levels, the anchor loads were normalized by their maximum value during a given test. The crack widths for each anchor \( w_j \) were normalized by the maximum crack width \( w_{\text{max},j} \) for the individual anchors. Normalized quantities are denoted by an asterisk. For example \( N^* \) and \( w^* \) refer to the normalized anchor load and normalized crack width, respectively.

Normalized anchor load at an instant of time “\( i \)” for anchor “\( j \)” is defined as follows:

\[
N^*_{i,j} = \frac{N_{i,j}}{N_{\text{max},j}}
\]

For the four anchor configuration tested herein, the anchor location \( j \) varies as NW, NE, SW, and SE to represent the North-West (4), North-East (2), South-West(3) and South-East (1) anchors respectively as shown in Figure 4.38. For example the North-West anchor at any discrete time instance “\( i \)” the normalized load is defined as:

\[
N^*_{i,NW} = \frac{N_{i,NW}}{N_{\text{max},NW}}
\]
The maximum load per load cycle is represented by the subscript “$k$”. The maximum normalized load per anchor $j$ per load cycle $k$ ($N_{\text{max},i,j}^*$) is defined as follows:

$$N_{\text{max},i,j}^* = \frac{N_{\text{max},i,j}}{N_{\text{max},j}}$$  \hspace{1cm} (4-6)

Normalized crack width at instant of time “$i$” for anchor “$j$” is defined as follows:

$$w_{i,j}^* = \frac{w_{i,j}}{w_{\text{max},j}}$$  \hspace{1cm} (4-7)

The maximum crack width per crack cycle is represented by the subscript “$k$”. Therefore, the maximum normalized crack width per anchor $j$ per crack cycle $k$ ($w_{\text{max},i,j}^*$) is defined as follows:

$$w_{\text{max},i,j}^* = \frac{w_{\text{max},k,i,j}}{w_{\text{max},j}}$$  \hspace{1cm} (4-8)
4.9 Data Post-Processing

The 83 channels of test data were post-processed using Matlab®. The post-processing in general terms included the following: synchronizing data from the UCSD and SDSU data acquisition systems, resampling all data to 200 Hz, filtering and baseline correction where appropriate, and calculation of various parameters of interest. It is noted that data was sampled on two different data acquisition (DAQ) systems (UCSD DAQ) and (SDSU DAQ). The UCSD DAQ samples at \( dt=0.004992 \) seconds (e.g. 200.3205 samples per second). The SDSU system samples at \( dt=0.005 \) seconds (e.g. 200 samples per second), this is compensated for in post-processing by resampling the UCSD data at 0.005 seconds.

The following steps were taken in the data post-processing:

- Data is read in from text files
- Find the timeshift between UCSD and SDSU data acquisition systems
  - Bias Slab Acceleration by the mean of the first one second of data UCSD (AccelN5, old name “A_NS_SLAB_N”) and SDSU (Accl1)
  - Resample Slab Acceleration for UCSD (AccelN5) and SDSU (Accl1) at 1000Hz
  - Find the timeshift between signals by using the cross-correlation of the two signals (Matlab command “xcorr”). Cross Correlation (also known as the sliding dot product) is defined as follows:

\[
(f \ast g)[n] = \sum_{m=-\infty}^{\infty} f^*[m]g[n+m] \quad (f \text{ and } g \text{ discrete})
\]

- Where \( f \) and \( g \) are continuous or discrete functions and \( f^* \) is the complex conjugate of \( f \). The concept is that the function \( f \) is slid in time, \( n \), along the function \( g \) and the integral of the product of the function of \( f \) and \( g \) is calculated for each
incremental amount of sliding (m). When the functions match up, the value of $f*g$ is at a maximum. The complex conjugate ($f^*$) assures that the integration is maximized for imaginary components of complex-valued functions.

- Resample UCSD data to 200 Hz. A “spline” interpolation routine is used.
- Apply the timeshift to the SDSU timevector (Channel 1) \( \text{newtime} = \text{time} - \text{timeshift} \) (a negative timeshift implies that the SDSU data lags the UCSD data, a positive timeshift implies that the SDSU data is ahead).
- All raw data is saved to Matlab binary format for ease of accessing with the extension “_raw.mat”.
- Raw Data is read in from the binary RAW files.
- Bias data
  - Acceleration channels are mean biased (subtract the mean of the entire record). Eliminates DC offset.
  - Load washer channels are not biased (load washers are not biased because they may have an initial non-zero value due to pretension/torque on the anchor).
  - All other data channels besides load washers and acceleration data are biased by subtracting the mean of the first one second of data.
- Filter data
  - Pad data to be filtered at the beginning and end with 10 seconds of zeros.
  - Accelerations: Apply a 3rd order Butterworth bandpass filter for acceleration channels using cutoff frequencies of 0.2 Hz to 50Hz. Use forward and backward filtering to eliminate filter delay (Matlab “filtfilt” command).
  - SAMU pins: Apply a 3rd order Butterworth low pass filter with 50Hz cutoff. Use forward and backward filtering to eliminate filter delay (Matlab “filtfilt” command).
All other channels have acceptable characteristics without filtering

- Unwanted quiescent data is cutoff from the beginning and end of the test. Guidance for cutoff locations “t,start” and “t,end” was provided by calculating the times corresponding to 1% and 99% Arias intensity (e.g. Arias, 1970) of the slab acceleration and add 3 seconds to beginning and end of data; however, final cutoff locations were adjusted by eye considering both input and response quantities, specifically, slab acceleration, slab displacement, WALLE acceleration, WALLE displacement, and anchor load response. where the Arias intensity is calculated as follows:

\[ I_A = \frac{\pi}{2g} \int_0^{T_d} a(t)^2 dt \]  

where \( I_A = \) Arias intensity of the acceleration time history, \( a(t) = \) acceleration due to gravity, \( t = \) time, and \( T_d = \) total duration of the record.

- Time vector is reinitialized so that the start of data is zero seconds (first data point is zero time)
- Save FILTERED data to .mat file in a folder called “FILTERED DATA”
- Read in FILTERED data and perform data analysis

### 4.9.1 Calculation of WALLE Displacement

Two system measurements of particular interest are the displacement and acceleration at the Center of Mass (CM), see Figure 4.39 of WALLE. No accelerometers or displacement transducers were located directly at the WALLE CG due to interference with the weight plates, therefore, this data had to be calculated from the neighboring sensors and kinematics of the system.
A schematic describing WALLE’s various displacement components is provided in Figure 4.39. Position 1 in this figure indicates the at rest position of the WALLE on the slab/CCILR assembly mounted on the shake table. During testing, the shake table platen moves relative to the strong wall to position 2. The table platen displacement in the North direction is herein called (Disp.Table). Simultaneously, the crack actuators are opening and closing cracks, which causes the slab to elongate due to crack opening and slab tension strains.

The total elongation of the slab (i.e. the displacement of the sliding end relative to the fixed end, Disp.Act) is measured by the cracking actuator displacement at the sliding end of CCILR relative to the fixed end of CCILR. The additional WALLE displacement at position 3, due to the slab elongation (Disp.Slab), is a fraction of Disp.Act where the fraction of total slab displacement at the centerline of WALLE is based on the WALLE position (A, B, or C) as shown in Figure 4.7. This fraction a is given in Table 4.16 and is herein called the position displacement ratio where a is the ratio of the distance from the fixed end to centerline of WALLE divided by the total length of the slab. The position displacement ratio a is given in Table 4.16. The height to the center of WALLE mass varied for the different weight configurations of WALLE and is given in Table 4.17. From these geometric parameters, the displacement of the WALLE at the height of the longitudinal string pot is calculated relative to the base of WALLE from the following equations (Disp.WALLE_{opt}). The relative WALLE displacement at the Center of Mass (Disp.WALLE_{CM}) is then calculated from the kinematics of deformation. It is noted that the WALLE displacement consists of two components (1) rigid body rotation of WALLE about the compression toe due to anchor deformation and (2) elastic bending of WALLE. The translation of the string pot displacement to the center of mass should only apply to the rigid body rotation component; however, a sensitivity study was performed on elastic deflection at the levels of anchor force encountered during dynamic testing and it was found that applying the correction factor to both components (rigid body rotation and elastic deflection) resulted in at least 98% accuracy of the true displacement.
Figure 4.39 Measurement of WALLE displacement

Table 4.16 Critical test setup dimensions

<table>
<thead>
<tr>
<th>WALLE Position</th>
<th>Distance from South end [in]</th>
<th>WALLE Position Ratio “a”</th>
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<tbody>
<tr>
<td>A</td>
<td>22</td>
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<tr>
<td>B</td>
<td>48</td>
<td>0.500</td>
</tr>
<tr>
<td>C</td>
<td>74</td>
<td>0.771</td>
</tr>
</tbody>
</table>

\[ a = \frac{\text{Distance from fixed end beam to WALLE centerline}}{\text{Length of Slab}} \]

\[ \text{Disp Slab} = (\text{Disp Act})a \]

\[ \text{Disp}_{\text{WALLE}}_{\text{spl.}} = \text{Disp.String.Pot} - \text{Disp.Slab} - \text{Disp.Table} \]

\[ \text{Disp}_{\text{WALLE}}_{\text{CM, stff}} = \text{Disp}_{\text{WALLE}}_{\text{spl.}} \left(\frac{38"}{h_{\text{splL}} = 48"}\right) = 0.792\text{Disp}_{\text{WALLE}}_{\text{spl.}} \]
\[ \text{Disp} WALLE_{CM, \text{flex}} = \text{Disp} WALLE_{\text{spl}} \left( \frac{55''}{h_{\text{spl}} = 48''} \right) = 1.146 \text{Disp} WALLE_{\text{spl}} \quad 4-14 \]

Table 4.17 WALLE properties: weight and height to center of mass

<table>
<thead>
<tr>
<th>WALLE Configuration</th>
<th>Total Weight [lbs]</th>
<th>Height from top of slab to C.M. ( h_{cm} ) [in]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flexible</td>
<td>2550</td>
<td>55</td>
</tr>
<tr>
<td>Stiff</td>
<td>860</td>
<td>38</td>
</tr>
</tbody>
</table>

4.9.2 Calculation of WALLE Acceleration

The WALLE CM varied for the WALLE configurations “stiff” and “flexible” therefore no accelerometer was located directly at the WALLE CM, resulting in the need to calculate the acceleration at the CM from neighboring acceleration measurements. A weighted linear interpolation of acceleration measurements, Accl3, Accl4, and Accl5 was used to calculated the acceleration at the center of mass as described in Figure 4.40. The following equations were used to calculate the acceleration of the CM for the stiff and flexible WALLE configurations:
Figure 4.40 Interpolation of WALLE acceleration at the center of mass

\[ a_{C.M,\text{stiff}} = a_4 \left( \frac{h_{C.M} - h_3}{h_4 - h_3} \right) + a_3 \left( \frac{h_4 - h_{C.M.}}{h_4 - h_3} \right) \]  \hspace{1cm} 4-15

\[ a_{C.M,\text{flexible}} = a_5 \left( \frac{h_{C.M} - h_4}{h_5 - h_4} \right) + a_4 \left( \frac{h_5 - h_{C.M.}}{h_5 - h_4} \right) \]  \hspace{1cm} 4-16

Where \( a_3, a_4, \) and \( a_5 \) are the measured acceleration time histories at Accl3, Accl4, and Accl5 respectively and \( a_{CM,\text{stiff}} \) and \( a_{CM,\text{flexible}} \) the calculated acceleration at the WALLE center of mass for the stiff and flexible WALLEs respectively.

4.9.3 Excerpt from Matlab® Code

The following is an excerpt of the Matlab code® that calculates the WALLE displacement and acceleration at the center of mass. First the code determines from the test name if the WALLE configuration is stiff or flexible and then the appropriate parameters are used in the subsequent calculations.
%% Determine Position of WALLE
if strcmp(walleloc{i},'C') == 1
    a = 0.771;
elseif strcmp(walleloc{i},'B') == 1
    a = 0.5;
elseif strcmp(walleloc{i},'A') == 1
    a = 0.229;
elseif strcmp(walleloc{i},'NA') == 1
    a = 0;
end

%% Calculate Displacement and Acceleration of WALLE at Center of Mass (CM)
% Calculate Intermediate Displacements
Disp1 = s.DispLon;   % Displacement at Long. String Pot (spL)
Disp2 = s.DispTab;   % Table Displacement
Disp3 = mean([s.ActEDisp s.ActWDisp],2);  % Slab elongation
% Input heights of Accelerometers on WALLE from base (in)
h3 = 8;   % Acc13
h4 = 48;  % Acc14 and longitudinal string pot
h5 = 73.5; % Acc15
% Calculate Information based on Stiff or Flexible WALLE
if ~isempty(regexp(FileName{i},'_F_', 'once'))
    hcm = 55; %height to CM for flexible WALLE (in)
    s.DispWALLE = (Disp1 - Disp2 - Disp3*a)*hcm/h4;
    s.Acc1WALLE = s.Acc15*((hcm-h4)/(h5-h4)) + s.Acc14*
        ((h5-hcm)/(h5-h4)); %linear interpolation between Acc14&5
    w = 2550; % Total weight of flexible WALLE (lbf)
elseif ~isempty(regexp(FileName{i},'_R_', 'once'))
    hcm = 38; %height to CM for stiff WALLE (in)
    s.DispWALLE = (Disp1 - Disp2 - Disp3*a)*hcm/h4;
    s.Acc1WALLE = s.Acc14*((hcm-h4)/(h4-h3)) + s.Acc13*
        ((h4-hcm)/(h4-h3)); % linear interpolation between Acc13&4
    w = 860; % Total weight of stiff "rigid" WALLE (lbf)
end
Chapter 5 Shake Table Test Program: System Characterization Results

5.1 Objectives

The purpose of this chapter is to characterize the anchored WALLE system in terms of system stiffness, natural period, and damping for the four anchor types under evaluation. This characterization, also known as system identification, is described in the following sections. The goal of the system identification, system ID, was to observe how anchor type and presence of cyclic cracking affect dynamic properties of an anchored NCS system, in this case natural frequency and damping. It is a well know phenomenon that anchor stiffness softens when anchors are located in statically open cracks, as compared to uncracked concrete, therefore the cyclic cracked case is a natural extension for investigation. To undertake this study, a series of system identification tests were performed on WALLE in each test setup in order to determine the natural frequency and damping of the system. The system ID tests included: 1) Modal impact hammer tests in uncracked concrete, 2) White noise tests in both uncracked and cracked concrete with static crack width of 0.8mm [0.03 in.]. Additionally, system ID was performed on the data from 3) Variable phase correlation tests in cyclic cracks having a maximum crack width of 0.8mm [0.03 in.], and 4) Failure tests in cyclic cracks. Before each test series, a modal impact hammer test was performed first, followed by white noise in uncracked concrete and in static cracks with crack width of 0.8mm [0.03 in.].

5.2 Modal Impact Hammer Tests

This section describes the results of the modal impact hammer tests. The modal impact hammer test system setup and instrumentation was described in detail in Chapter 4. A hammer
test was performed on the WALLE at the beginning of each new WALLE setup (e.g., for each different anchor type failure tests and before the variable phase correlation tests). The hammer tests were conducted only on the uncracked concrete boundary condition. This limitation is due to the fact that the crack actuators have to be active to hold the cracks open, therefore, for safety concerns, people were not allowed stand on the concrete slab to perform hammer tests while the cracks were open and actuators were active. Moreover, impact hammer tests on the cracked case were not necessary, because it was observed that rocking of the WALLE was not induced enough to overcome the 2550 lb dead weight of the WALLE. In other words, the anchors were not engaged sufficiently during the low amplitude hammer tests to determine if the frequency would change when the anchor was located in a crack or not in a crack.

The impact hammer system was used in a single-input multiple-output (SIMO) configuration. In this case, the hammer acts as the applied input and the reference accelerometers as the multiple outputs. This application of multiple reference accelerometers is suitable for capturing the first few modes of a system. The testing equipment allows for measurement of the impact hammer force as well as the time histories of each accelerometer. To obtain the natural frequency and damping of the equipment, multiple transfer functions are created by transforming the signal into the frequency domain using the Fast Fourier Transform (FFT). The magnitude of the FFT of the output signal is then divided by that of the impact signal to create a transfer function, which is also known as a Frequency Response Function (FRF).

A bandpass filter is applied to the FRF for frequencies ranging from about 2 Hz to 50 Hz. ME’Scope (www.vibetech.com) utilizes an orthogonal polynomial, a refraction polynomial method, to fit the FRF for a specified bandwidth (Schwarz and Richardson, 1999). Using the FRF trace, the peaks are an indication of the natural frequencies. Damping is then calculated at a given resonant frequency using the half-power bandwidth method. The half-power bandwidth method
takes the ratio of the frequency bandwidth associated with \(1/(\sqrt{2})\) times the peak amplitude over two times the resonant frequency of interest.

The FRF of an average of multiple impacts at the same location was used to increase the reliability of the identified peaks at the natural frequencies. In addition, from multiple impacts, coherence curves for each accelerometer were calculated and evaluated. The coherence is an indicator of the robustness of the impact as it measures how much of the response is due to the input force. It is typically plotted as a function of frequency and if greater than 0.9 (90%) at and around each natural frequency one may consider the measurement as highly reliable. A coherence of 1.0 (100%) indicates that all of the response is due to the impact.

It should be noted that the vibration level during an impact hammer test is relatively low. The load levels would be incompatible with higher levels of damping and potential elongation of the natural periods expected during even minor earthquake excitation. Nonetheless, this data provides an indication of the in-situ low amplitude dynamic characteristics of the anchored model NCS. Table 5.1 summarizes the damping and natural period results for all impact hammer tests. The average WALLE period and damping obtained from the hammer tests was 0.21 sec. (0.58% damping) for the flexible WALLE and 0.097 sec. (0.42% damping) for the stiff WALLE configuration. These period values are very close to the targets of 0.25 and 0.1 seconds.
Table 5.1 WALLE fundamental period and damping (impact hammer tests)

<table>
<thead>
<tr>
<th>Date</th>
<th>Anchor Type</th>
<th>$T_{\text{WALLE,uncr}}$ [s]</th>
<th>$f_{\text{WALLE,uncr}}$ [Hz]</th>
<th>Damping $\xi_{\text{uncr}}$ [% critical]</th>
</tr>
</thead>
<tbody>
<tr>
<td>12/11/09</td>
<td>Epoxy (1000 lb tension)</td>
<td>0.211</td>
<td>4.74</td>
<td>0.04%</td>
</tr>
<tr>
<td>11/04/09</td>
<td>Expansion (2500 lb tension)</td>
<td>0.209</td>
<td>4.79</td>
<td>0.96%</td>
</tr>
<tr>
<td>11/04/09</td>
<td>Expansion (0 lb tension)</td>
<td>0.197</td>
<td>5.08</td>
<td>0.54%</td>
</tr>
<tr>
<td>01/27/10</td>
<td>Expansion (1000 lb tension)</td>
<td>0.212</td>
<td>4.71</td>
<td>--</td>
</tr>
<tr>
<td>01/20/10</td>
<td>Drop-In</td>
<td>0.235</td>
<td>4.25</td>
<td>0.20%</td>
</tr>
<tr>
<td>10/30/09</td>
<td>V1 Undercut (50 lb tension)</td>
<td>0.210</td>
<td>4.76</td>
<td>0.99%</td>
</tr>
<tr>
<td>12/04/09</td>
<td>V2 Undercut (50 lb tension)</td>
<td>0.215</td>
<td>4.65</td>
<td>0.75%</td>
</tr>
<tr>
<td></td>
<td>Average Flexible WALLE:</td>
<td>0.21</td>
<td>4.7</td>
<td>0.58%</td>
</tr>
<tr>
<td>10/28/09</td>
<td>V1 Undercut (50 lb bolt tension)</td>
<td>0.0952</td>
<td>10.5</td>
<td>0.22%</td>
</tr>
<tr>
<td>10/28/09</td>
<td>V1 Undercut (1000 lb tension)</td>
<td>0.0935</td>
<td>10.7</td>
<td>0.10%</td>
</tr>
<tr>
<td>10/29/09</td>
<td>V1 Undercut (50 lb tension)</td>
<td>0.1031</td>
<td>9.7</td>
<td>0.66%</td>
</tr>
<tr>
<td>10/30/09</td>
<td>V1 Undercut (50 lb tension)</td>
<td>0.0980</td>
<td>10.2</td>
<td>0.55%</td>
</tr>
<tr>
<td>12/03/09</td>
<td>V2 Undercut (50 lb bolt tension)</td>
<td>0.0971</td>
<td>10.3</td>
<td>0.56%</td>
</tr>
<tr>
<td></td>
<td>Average Stiff WALLE:</td>
<td>0.097</td>
<td>10.3</td>
<td>0.42%</td>
</tr>
</tbody>
</table>

5.3 Analysis Model of WALLE

For comparison with experimental results, a linear elastic Finite Element Analysis Model (FEA/FEM) was developed. The objective of the model was to determine the elastic natural periods of vibration and mode shapes of the WALLE for the following cases: rigid anchorage and flexible anchorage considering anchors with varying linear elastic initial stiffness ($K_i$). The model consisted of linear elastic beam elements for the WALLE and weight plates and elastic axial spring elements for the anchors. For the rigid anchorage case, pinned supports were used at the four anchor locations. The actual behavior of the flexible anchorage case is a more complicated non-linear problem, which includes a gap opening beneath the SAMU, a compression only spring for the concrete and tension only spring for the anchorage. This was beyond the scope of this simple model for comparison/verification of elastic natural period and modes shapes, therefore in the flexible anchor model, the compression toe was considered as a
pinned support and the anchor was considered as an axial tension/compression spring of equal compression/tension initial stiffness ($K_i$).

The flexible and stiff WALLE configurations were modeled. The first two mode shapes in the direction of motion for the flexible WALLE configuration are shown in Figure 5.1. The first two mode shapes in the direction of motion for the stiff WALLE configuration are shown in Figure 5.2. In both cases, the mode one is typical fixed cantilever bending behavior with all DOF in-phase, whereas mode two is rotation of the mass plates about an axis perpendicular to the page. The later results in out-of-phase behavior for the DOFs associated with measurements at Accl5 and Accl4. The experimentally observed mode two period was approximately 0.028 seconds (35 Hz). The modal observations are consistent with the experimental behavior of the WALLE during the white noise tests. Furthermore, these models provide verification for the data post-processing assumption that the acceleration at the center of mass (CM) can be considered as a linear interpolation of the adjacent accelerometers, weighted solely based on their distance from the CM.
**Figure 5.1** Flexible WALLE mode shapes and modal periods (rigid anchors)

**Mode 1** – 5.0 Hz [0.20 s]  
Bending (In-Plane)

**Mode 2** – 35.2 Hz [0.0284 s]  
Rotation of mass


**Figure 5.2** Stiff WALLE mode shapes and modal periods (rigid anchors)

**Mode 1** – 14 Hz [0.071 sec]  
Bending

**Mode 2** – 105 Hz [0.00952 sec]  
Rotation of mass
5.4 White Noise Tests

Using the procedures and input motions described previously, Gaussian white noise tests were performed to determine the WALLE natural period and damping in uncracked and cracked concrete for various anchor types. The system ID processing methods and results as described in the following sections.

5.4.1 Natural Period of Vibration

The data processing procedure for obtaining the natural period of vibration of the anchored WALLE system was the same for both the white noise tests and the seismic tests, only the input motions varied. The Frequency Response Function (FRF) was determined using the following techniques: (1) Transfer Function Estimate using power spectral density, herein abbreviated as “TF PSD”, (2) Transfer Function Estimate using spectral acceleration, herein abbreviated as “TF Sa” and (3) Transfer Function Estimate using a dynamic model, herein abbreviated as “TF Model”. All methods use the acceleration as measured at the slab as the input signal and the acceleration calculated at the WALLE center of mass as the output signal.

The transfer function using the power spectral density used the Matlab® function \( T_{xy} = \text{tfestimate}(x,y) \) where \( x \) is the input vector and \( y \) is the output vector. The relationship between \( x \) and \( y \) is modeled by the linear time-invariant transfer function \( T_{xy} \) where \( T_{xy} \) is the cross power spectral density \( (P_{xy}) \) of \( x \) and \( y \) and the spectral density of \( x \) \( (P_{xx}) \) as given by the equation below. The power spectral density uses Welch’s method (Welch, 1967).

\[
T_{xy}(f) = \frac{P_{xy}(f)}{P_{xx}(f)} \tag{5-1}
\]

The transfer function was also calculated by a second method using the ratio of the 2% damped spectral acceleration of the output divided by the input. A damping of 2% was chosen
based on the recommendations of ASCE 43, 2005 and results of the transfer function using the power spectral density for the white noise tests which is presented later.

\[ T_{xy,sa}(f) = \frac{Sa_y(f)}{Sa_x(f)} \]  

5-2

The transfer function was also calculated by a third method that uses a simulation model of a single input single output (SISO) dynamic system. A full description of the method is beyond the scope of this report; however, the model uses a Matlab® model of the dynamic system, using the following commands, `iddata`, `pem`, `sim`. Then, the transfer function estimate is calculated from the results of the SISO model that fits the WALLE experimental data.

The transfer function using all three methods is plotted below for a white noise test in uncracked (Figure 5.3) and cracked concrete (Figure 5.4). The transfer function plots were examined in detail for all experimental tests. In general, the spectral acceleration method yields the smoothest curves, but the peaks tend to be wider than the other two methods, which affects the calculation of damping as discussed in a later section. It is noted that the frequency at which the peak of the transfer function occurs is the natural frequency of the system. The natural frequency using the power spectral density and the spectral acceleration tended to produce similar shapes and closely aligned natural frequencies. The simulation model corresponded well for the white noise tests; however, for earthquake input motions, sometimes the model transfer function did not match well or could not be determined at all. Based on the above factors, it was determined that the best method for the determination of WALLE period and damping was the transfer function estimate using the power spectral density. This is also the most common method in signal processing applications and has therefore been selected for use in all further estimations of WALLE damping and period.
Figure 5.3 Example transfer function plot stiff WALLE (white noise, uncracked, test #1, WALLE.R.N9.UC.WN) f=10 Hz \(T=0.10\) s.
Figure 5.4 Example transfer function plot stiff WALLE (white noise, cracked, static \(w=0.8\text{mm}\), test #2, WALLE.R.N9.CC08.WN) \(f=9.09\text{ Hz}\) \([T=0.11\text{ sec.}]\)

Table 5.2 lists the test number and natural period of vibration of flexible WALLE in uncracked concrete \(T_{\text{WALLE,uncr}}\) and cracked concrete boundary conditions with a static crack width of 0.8 mm \(T_{\text{WALLE,cr}}\) for the various anchor types during experimental white noise tests. The period of vibration ranges from 0.216 to 0.316 seconds (3.2 to 4.6 Hz) for the uncracked case and 0.242 to 0.316 seconds (3.7 to 4.1 Hz) for the cracked case. The average period of flexible WALLE in static cracked concrete was 0.26 sec. On average, the flexible WALLE period elongates by 6.2% in the cracked case as compared to the uncracked case. Due to the square root relationship between period and stiffness, this 6.2% elongation in period corresponds to a reduction in system stiffness of 13% when the anchors are located in the cracked boundary condition of 0.5 or 0.8mm [0.03 in.] wide cracks.
Table 5.2 Flexible WALLE fundamental period for various anchor types (white noise tests)

<table>
<thead>
<tr>
<th>Test No. (uncracked/cracked)</th>
<th>Anchor Type</th>
<th>$T_{\text{WALLE,uncr}}$ [s]</th>
<th>$T_{\text{WALLE,cr}}$ [s]</th>
<th>$f_{\text{WALLE,uncr}}$ [Hz]</th>
<th>$f_{\text{WALLE,cr}}$ [Hz]</th>
</tr>
</thead>
<tbody>
<tr>
<td>127 / 128</td>
<td>Epoxy ($w = 0.8$ mm)</td>
<td>0.222</td>
<td>0.251</td>
<td>4.50</td>
<td>3.98</td>
</tr>
<tr>
<td>142 / 143</td>
<td>Expansion ($w = 0.5$ mm)</td>
<td>0.216</td>
<td>0.245</td>
<td>4.63</td>
<td>4.08</td>
</tr>
<tr>
<td>52 / 53</td>
<td>Expansion ($w = 0.8$ mm)</td>
<td>0.218</td>
<td>0.273</td>
<td>4.59</td>
<td>3.66</td>
</tr>
<tr>
<td>133 / 134</td>
<td>Drop-In ($w = 0.8$ mm)</td>
<td>0.316</td>
<td>0.271</td>
<td>3.16</td>
<td>3.69</td>
</tr>
<tr>
<td>114 / 115</td>
<td>Undercut ($w = 0.8$ mm)</td>
<td>0.235</td>
<td>0.242</td>
<td>4.26</td>
<td>4.13</td>
</tr>
<tr>
<td>AVERAGE:</td>
<td></td>
<td>0.241</td>
<td>0.256</td>
<td>4.15</td>
<td>3.91</td>
</tr>
</tbody>
</table>

Figure 5.5 illustrates the period elongation that occurs for stiff WALLE when the uncracked case is compared with the cracked case where the static crack width was 0.8 mm (0.03 in) for N9 undercut anchors. The average stiff WALLE period is 0.10 and 0.13 seconds in uncracked concrete and static cracks respectively. This is on average a 25% increase in period which corresponds to a 56% decrease in system stiffness. Figure 5.6 illustrates the period elongation that occurs for flexible WALLE when the uncracked case is compared with the cracked case where the static crack width was 0.8 mm (0.03 in) for all anchor types. The average flexible WALLE period is 0.23 and 0.26 seconds in uncracked concrete and static cracks having a width of 0.8 mm respectively. The average period elongation for all cases is 13% which corresponds to a 28% decrease in system stiffness. The lesser stiffness change for the flexible versus stiff WALLE in cracked versus uncracked concrete may be due to the fact that the flexible WALLE has a higher P-delta geometric stiffness due to the larger mass (2550 versus 860 lbs), which could be a large contributor to stiffness at the relatively low levels of vibration induced during the white noise tests. Period elongation occurs for the cracked case for all anchors except N7, drop-in anchors, which is likely an anomaly due to the variations in the period calculation when the FRF is not a smooth function and has multiple and distinct peaks in the vicinity of the natural period.
Figure 5.5 Period of stiff WALLE obtained from white noise tests

Figure 5.6 Period of flexible WALLE obtained from white noise tests
Figure 5.7 summarizes the WALLE natural period obtained from all white noise tests. It is of interest to note that WALLE period is not sensitive to different anchor types at the low to moderate motion intensity used in the white noise tests (0.03g RMS). WALLE period is more sensitive to cracked or uncracked concrete conditions than it is to anchor type. It is further noted that a contributor to the period elongation may be caused by the reduction in bending stiffness of the slab due to the opening of discrete cracks in the slab. The mean period in uncracked and cracked concrete are defined as $T_{m,\text{uncr}}$ and $T_{m,\text{cr}}$ respectively and the subscripts flex and stiff denote flexible and stiff WALLE respectively.

![Figure 5.7 Period of WALLE obtained from all white noise tests](image)

Figure 5.8 summarizes the fundamental period of vibration of WALLE obtained during the white noise tests related to the anchor initial stiffness obtained from the single anchor reference tests. As expected, there is a general trend that WALLE period elongates as anchor initial stiffness decreases. Also plotted are the results of a finite element Eigenvalue analysis of
the flexible WALLE with varying anchor stiffness. The finite element model was described in the previous section. The model predicts the WALLE period well for anchors with large stiffness \((K_i = 100-500 \text{ kip/in})\), but tends to over predict the WALLE period at very low anchor stiffness. This is likely because the Eigenvalue analysis does not take into account the dead weight of the WALLE which in the experimental case reduces the WALLE period at low intensity floor motion white noise tests because of the resistance to overturning (stiffening P-delta effect) provided by the WALLE dead weight. Until the static overturning of the WALLE dead weight is overcome the anchors are not engaged (i.e. in the experimental case, the anchor stiffness is not engaged as part of the system until the compression between the SAMU foot and the concrete is overcome).

![Figure 5.8 WALLE fundamental period (white noise test series)](image)

5.4.2 System Damping

Damping was then calculated from the transfer function (TF) using the half-power bandwidth method (e.g. Clough and Penzien, 2003). In this method, the damping ratio is
determined from the frequencies at which the response amplitude $A$ is reduced to the level $1/\sqrt{2}$ times its peak value $A_{\text{max}}$. Because the peak is narrow, it was necessary to resample the TF at finely spaced frequencies near the maximum amplitude. Damping is then calculated by the following equation:

$$\xi = \frac{f_b - f_a}{f_b + f_a}$$

Where $f_a$ and $f_b$ are the frequencies at which the amplitudes of response equal $1/\sqrt{2}$ times the maximum amplitude. Figure 5.9 summarizes the damping ratio obtained from the stiff WALLE during white noise tests. The general trend is that the damping ratio is larger for WALLE anchored in cracked concrete as opposed to uncracked concrete. Figure 5.10 summarizes the damping ratio obtained from the flexible WALLE during white noise tests. Similarly, there is a general trend that the damping ratio is larger for WALLE anchored in cracked concrete as opposed to uncracked concrete for the epoxy and expansion anchors; however, the trend is the opposite for the drop-in and undercut anchors where the damping ratio decreases in cracked concrete. Figure 5.11 summarizes the damping ratio obtained from all white noise tests, all anchor types, and both cracked and uncracked concrete. The average damping ratio was 1.8%, with a coefficient of variation of 64% indicating a large variation in damping. Based on the average damping result of 1.8% damping, the spectra throughout this report are calculated using a damping ratio of 2%. Interestingly, a damping ratio of 2% is recommended by the American Society of Civil Engineers (ASCE 43, 2005) for massive mechanical components such as pumps, compressors, fans, and motors for response level one. Response level one corresponds to a demand to capacity ratio (D/C) equal to 0.50 or less (i.e. components that are not highly stressed and expected to remain elastic). The D/C ratios were targeted as less than 0.50 for the white noise tests and were on average approximately 0.10, where the demand is the maximum anchor load
experience by each anchor during the test and the capacity is the code design capacity of the anchor in cracked or uncracked concrete using the procedures found in the ICC-ESR reports (ICC-ESR 1917, 2007; ICC-ESR 1546, 2008; ICC ESR 2322, 2010; and ICBO-ER 1372, 2000) for the anchor type under consideration.

Figure 5.9 Damping ratio of stiff WALLE obtained from white noise tests
Figure 5.10 Damping ratio of flexible WALLE obtained from white noise tests

- Uncracked
- Cracked

Period, s

Uncracked
- Exp 
- Drop-In

Cracked
- Exp 
- Drop-In

Epoxy
- W=0.8mm
- W=0.5mm

Expansion
- W=0.8mm

Undercut
- W=0.8mm

Drop-In

- Exp 
- Drop-In

Epoxy
- W=0.8mm
- W=0.5mm

Expansion
- W=0.8mm

Undercut
- W=0.8mm

- Exp 
- Drop-In

Epoxy
- W=0.8mm
- W=0.5mm

Expansion
- W=0.8mm

Undercut
- W=0.8mm

- Exp 
- Drop-In

Epoxy
- W=0.8mm
- W=0.5mm

Expansion
- W=0.8mm

Undercut
- W=0.8mm

- Exp 
- Drop-In

Epoxy
- W=0.8mm
- W=0.5mm

Expansion
- W=0.8mm

Undercut
- W=0.8mm

- Exp 
- Drop-In

Epoxy
- W=0.8mm
- W=0.5mm

Expansion
- W=0.8mm

Undercut
- W=0.8mm

- Exp 
- Drop-In

Epoxy
- W=0.8mm
- W=0.5mm

Expansion
- W=0.8mm

Undercut
- W=0.8mm

- Exp 
- Drop-In

Epoxy
- W=0.8mm
- W=0.5mm

Expansion
- W=0.8mm

Undercut
- W=0.8mm

- Exp 
- Drop-In

Epoxy
- W=0.8mm
- W=0.5mm

Expansion
- W=0.8mm

Undercut
- W=0.8mm

- Exp 
- Drop-In

Epoxy
- W=0.8mm
- W=0.5mm

Expansion
- W=0.8mm

Undercut
- W=0.8mm

- Exp 
- Drop-In

Epoxy
- W=0.8mm
- W=0.5mm

Expansion
- W=0.8mm

Undercut
- W=0.8mm

- Exp 
- Drop-In

Epoxy
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- W=0.5mm

Expansion
- W=0.8mm

Undercut
- W=0.8mm

- Exp 
- Drop-In

Epoxy
- W=0.8mm
- W=0.5mm

Expansion
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Undercut
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- Exp 
- Drop-In

Epoxy
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- W=0.5mm

Expansion
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Undercut
- W=0.8mm

- Exp 
- Drop-In

Epoxy
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- W=0.5mm

Expansion
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Undercut
- W=0.8mm

- Exp 
- Drop-In

Epoxy
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- W=0.5mm

Expansion
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Undercut
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- Exp 
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- Drop-In

Epoxy
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- Exp 
- Drop-In

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- W=0.5mm

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Undercut
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- Exp 
- Drop-In

Epoxy
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- W=0.5mm

Expansion
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Undercut
- W=0.8mm

- Exp 
- Drop-In

Epoxy
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- W=0.5mm

Expansion
- W=0.8mm

Undercut
- W=0.8mm

- Exp 
- Drop-In

Epoxy
- W=0.8mm
- W=0.5mm

Expansion
- W=0.8mm

Undercut
- W=0.8mm

- Exp 
- Drop-In

Epoxy
- W=0.8mm
5.5 Variable Phase Correlation Tests

5.5.1 Natural Period of Vibration

The WALLE natural period of vibration was calculated by finding the period corresponding the maximum of the frequency response function for the variable phase correlation tests using each of the 20 input motions (FM01-20). This is the same procedure as performed for the white noise tests. The average period of the flexible and stiff WALLE was 0.26 and 0.12 seconds, respectively (Figure 5.12). The period remained very consistent even though the correlation input motions had very different characteristics in terms of frequency content and amplitude. This indicates the system remains relatively elastic under these motions. Furthermore, the WALLE period was consistent between the version 1 and 2 test series, which demonstrates the repeatability of the anchored system under multiple installations.
5.5.2 System Damping

The WALLE damping ratio was calculated by the half-power bandwidth of the frequency response function for the variable phase correlation tests. This is the same procedure as performed for the white noise tests. The mean damping ratio, $\zeta_m$, over all correlation tests of the flexible and stiff WALLE was 3.8% and 1.5% of critical respectively (Figure 5.13). It is noted that there is a wide scatter in the damping ratio calculated from the various tests. The maximum input acceleration for the variable phase correlation tests ranged from 0.24 g to 1.1 g and the maximum WALLE response acceleration at the center of mass ranged from 0.24 g to 2.8 g. The WALLE damping ratios are sensitive to the amplitude of motion, therefore the average total damping ratio of 4% applies to an anchored flexible WALLE under this range of motion.
amplitude. This is larger than the 2% damping obtained from white noise tests because the anchors become engaged by larger tension loads during the correlation tests.

![Damping Ratio Graph](image)

**Figure 5.13 WALLE damping ratio (variable phase correlation test series)**

### 5.6 Unanchored WALLE Rocking Tests

The WALLE has two limiting configurations, the first is completely rigid anchors, the second is unanchored. As the anchors deform and lose strength and stiffness under increasing motion amplitude, the WALLE tends towards the limit case of being unanchored. Unanchored rocking tests were undertaken to characterize the period of WALLE. In the first set of tests, WALLE was manually rocked back and forth with a sinusoidal forcing function corresponding to the natural frequency, while the vertical displacement of the SAMUs were rocked to a constant peak displacement amplitude of 0.06”, 0.13”, 0.25”, and 0.5” respectively. The WALLE period was determined from the FFT of the experimental acceleration response for the various
amplitudes of rocking input (Table 5.3). The experimentally determined period was compared with the theoretical solution of an unanchored rigid block (ASCE 43, 2005) and the periods were found to be very similar (less than 9% difference). The frequency of WALLE anchored in uncracked concrete with expansion anchors from impact hammer tests was 4.8Hz (0.21 sec). The frequency drops to 3.5Hz (0.29 sec) which is 72% of the initial anchored frequency with only a small displacement of the SAMUs of 0.063in (1.6 mm) (e.g. Figure 5.14). Larger anchor displacements typically associate with anchor failure 0.25 to 0.5in (6.3 to 13mm) cause a reduction in frequency to 20 to 40% of the initial frequency. This drastic period reduction shift the WALLE significantly on the floor spectra. In the case of the reference motion FM02 the period elongation shifts WALLE to the right on the spectrum over and off of the peak spectral acceleration of 4.8g down to 0.5 to 1.5g (Figure 5.15).

Several unanchored tests were also conducted using earthquake input motions. A subset of six of the correlation motions were chosen for this test series (Table 5.4). The peak input acceleration for these tests ranged from 0.37 to 0.75g. The predominant period of input motion, calculated by the peak of the absolute value of the magnitude of the FFT of the floor input acceleration, varied from 0.10 to 0.93 sec; however, the WALLE period was between 0.21 and 0.39 sec demonstrating that WALLE period of rocking is independent of the input motion predominant period.

Table 5.3 Flexible WALLE unanchored rocking period (harmonic input)

<table>
<thead>
<tr>
<th></th>
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<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>VERY LOW</td>
<td>0.2</td>
<td>0.063</td>
<td>0.179</td>
<td>0.29</td>
<td>0.30</td>
<td>2.6%</td>
</tr>
<tr>
<td>LOW</td>
<td>0.3</td>
<td>0.125</td>
<td>0.358</td>
<td>0.42</td>
<td>0.42</td>
<td>0.5%</td>
</tr>
<tr>
<td>MED</td>
<td>0.7</td>
<td>0.250</td>
<td>0.716</td>
<td>0.55</td>
<td>0.60</td>
<td>8.8%</td>
</tr>
<tr>
<td>HIGH</td>
<td>1.4</td>
<td>0.500</td>
<td>1.433</td>
<td>0.93</td>
<td>0.87</td>
<td>-6.6%</td>
</tr>
</tbody>
</table>
Figure 5.14 WALLE frequency of unanchored rocking versus amplitude of rigid body rocking rotation, theoretical versus measured

Figure 5.15 Effect of WALLE period elongation due to anchor axial displacement ($\delta$) on spectral acceleration at 5% damping for reference floor motion FM02 at 100% scale
Table 5.4 Flexible WALLE unanchored rocking period (earthquake input)

<table>
<thead>
<tr>
<th>Test Name</th>
<th>Peak Input Acceleration [g]</th>
<th>Sa(T=0.25sec, ζ=2%) [g]</th>
<th>Predominant Period T_p,input [s]</th>
<th>WALLE Period T_WALLE [s]</th>
</tr>
</thead>
<tbody>
<tr>
<td>V2.WALLE.F.UA.CC08.FM20</td>
<td>0.43</td>
<td>0.41</td>
<td>0.6</td>
<td>0.21</td>
</tr>
<tr>
<td>V2.WALLE.F.UA.CC08.FM13</td>
<td>0.45</td>
<td>0.42</td>
<td>0.1</td>
<td>0.26</td>
</tr>
<tr>
<td>V2.WALLE.F.UA.CC08.FM10</td>
<td>0.37</td>
<td>0.50</td>
<td>0.93</td>
<td>0.37</td>
</tr>
<tr>
<td>V2.WALLE.F.UA.CC08.FM05</td>
<td>0.51</td>
<td>1.20</td>
<td>0.24</td>
<td>0.36</td>
</tr>
<tr>
<td>V2.WALLE.F.UA.CC08.FM04</td>
<td>0.68</td>
<td>1.32</td>
<td>0.13</td>
<td>0.39</td>
</tr>
<tr>
<td>V2.WALLE.F.UA.CC08.FM03</td>
<td>0.75</td>
<td>3.00</td>
<td>0.12</td>
<td>0.32</td>
</tr>
<tr>
<td>MIN</td>
<td>0.37</td>
<td>0.41</td>
<td>0.10</td>
<td>0.21</td>
</tr>
<tr>
<td>MAX</td>
<td>0.75</td>
<td>3.00</td>
<td>0.93</td>
<td>0.39</td>
</tr>
<tr>
<td>AVERAGE</td>
<td>0.53</td>
<td>1.14</td>
<td>0.35</td>
<td>0.32</td>
</tr>
</tbody>
</table>

5.7 Summary Remarks

Based on the system identification experiments, the following observations can be made:

- Damping is dependent on the amplitude of vibration; therefore, the damping calculated from the low-level impact hammer tests, as expected, is lower than the damping calculated during the white noise and correlation tests. The average damping for the flexible WALLE from the hammer tests was 0.58% while the damping from the white noise tests and correlation tests was 1.8% and 3.8%, respectively. Based on these experimental results, a damping of 2% of critical is used in all subsequent analyses.

- The experimentally determined damping level of 2% corresponds well to published recommended values for damping in the literature for lightly stressed mechanical nonstructural components having heavy steel construction similar to the WALLE.

- The natural period of the anchored WALLE system was relatively independent of anchor type during the low amplitude white noise tests for both the uncracked and cracked concrete cases.
• The natural period of the anchored WALLE system was dependent on the boundary conditions of the concrete slab (uncracked versus static cracked) during the low amplitude white noise tests. On average there was an average period elongation of approximately 6% when the anchors were located in statically open cracks having a width of 0.8 mm. This corresponds to a reduction in system stiffness of 13% when the anchors are located in the cracked boundary condition of 0.8mm [0.03 in.] wide cracks as compared to closed cracks (uncracked case). It is not conclusive however if the change in system stiffness is entirely due to a change in anchor stiffness in open cracks, as the flexural stiffness of the slab may also decrease when static cracks are opened, which could also lead to a reduction in overall stiffness of the anchored WALLE system.

• The flexible WALLE period averaged 0.23, 0.26, 0.26, and 0.21 seconds for the cases of white noise uncracked, white noise cracked, correlation tests and hammer tests respectively. The hammer test period is lower likely due to the low level of impact amplitude and uncracked concrete used in the tests. The stiff WALLE period averaged 0.10, 0.13, and 0.12 seconds for the cases of white noise uncracked, white noise cracked and correlation tests, respectively. The period of the WALLE did not appear to be sensitive to whether the cracked case was static cracks at 0.8mm width (white noise tests) or cycled cracked with a maximum width of 0.8mm (correlation tests).

• The flexible WALLE period was less elongated in cracked concrete versus uncracked concrete than the stiff WALLE when located in static 0.8mm wide cracks during the white noise tests. This is likely due to the larger contribution of P-Delta geometric stiffness provided by the larger mass of the flexible WALLE (2550 versus 860 lbs), (i.e. the anchor flexibility does not become engaged in
tension until the compression between the SAMUS and the concrete due to the self weight of WALLE is overcome by rocking).

- It was observed from unanchored rocking tests that the WALLE period elongates dramatically when unanchored as compared to the fully anchored case. This has ramifications in the higher amplitude failure tests that as the anchors displace and loose stiffness, the WALLE period shifts significantly to a different region of the floor motion spectrum. In other words, floor acceleration response spectra often have dramatic amplitude changes with small changes in period, therefore, any small change in system stiffness can result in significant changes in demand. Additionally, there was good correlation between experimental and theoretical results for unanchored rocking period.
Chapter 6 Shake Table Test Program: Anchor Response

Results

6.1 Introduction

This chapter focuses on the anchor level results from the WALLE dynamic shake table failure tests in terms of load-displacement-crack width ($N, \delta, w$) and comparison of dynamic and monotonic test results. In the failure tests, the WALLE was subjected to the reference floor motion, FM02 at increasing levels of input acceleration until anchor failure occurred. The input motions and amplitude levels are described in full detail in Chapter 5. A summary of data from each test including plots of input and response is located in Appendix C. Two of the tests discussed were performed by Hoehler and Dowell (2010) in uncracked concrete. The test results are included to provide for a comparison between behavior in uncracked concrete and concrete with cyclic cracks.

Test videos were produced for each of the failure tests (Table 6.1). The videos include overall WALLE response and anchor response that have been time synchronized with data plots of input and response quantities such as: input acceleration, input displacement, crack width, anchor load, and anchor displacement (Figure 6.1). The videos allow for the synthesis of results from the test such as load and crack width amplitude at the time of anchor failure. The videos are also the first time to the author’s knowledge that anchor failure in cyclically cracked concrete due to inertial seismic loading has been captured with corresponding instrumentation.
Table 6.1 Failure test video summary

<table>
<thead>
<tr>
<th>Anchor</th>
<th>Test No</th>
<th>Test Name</th>
<th>Date of Test</th>
</tr>
</thead>
<tbody>
<tr>
<td>Epoxy</td>
<td>132</td>
<td>WALLE_F_N4_CC08_FM02_30P</td>
<td>12/10/2009</td>
</tr>
<tr>
<td>Expansion</td>
<td>126</td>
<td>WALLE_F_N5_CC05_FM02_100P</td>
<td>12/09/2009</td>
</tr>
<tr>
<td></td>
<td>146</td>
<td>V2_WALLE_F_N5_CC05_FM02_100P</td>
<td>01/27/2010</td>
</tr>
<tr>
<td></td>
<td>57</td>
<td>WALLE_F_N5_CC08_FM02_125P</td>
<td>11/04/2009</td>
</tr>
<tr>
<td>Drop-In</td>
<td>141</td>
<td>WALLE_F_N7_CC08_FM02_100P</td>
<td>01/25/2010</td>
</tr>
<tr>
<td>Undercut</td>
<td>51</td>
<td>WALLE_F_N9_CC08_FM02_60P</td>
<td>11/02/2009</td>
</tr>
<tr>
<td></td>
<td>120</td>
<td>V2_WALLE_F_N9_CC08_FM02_60P</td>
<td>12/07/2009</td>
</tr>
<tr>
<td>Undercut</td>
<td>176</td>
<td>STRUCT_N9_CC08_CP2_D1c</td>
<td>02/17/2010</td>
</tr>
<tr>
<td></td>
<td>179</td>
<td>STRUCT_N9_CC08_CR14_D14c</td>
<td>02/17/2010</td>
</tr>
</tbody>
</table>

Figure 6.1 Screen capture of a typical video produced for failure tests with synchronized data and video (Test 57: WALLE.F.N5.CC08.FM02.125P)
6.2 Anchor Performance in Uncracked and Cyclic Cracked Concrete (Comparison of SDSU and UCSD Tests)

Two of the anchors in this program (epoxy and expansion anchors) were tested in both uncracked and cyclic cracked concrete to allow for a direct comparison of anchor performance in the two extreme concrete boundary conditions. The uncracked case was tested at SDSU as a separate portion of the testing program; however, results are repeated herein to facilitate comparison with the cracked concrete boundary case tests, which were conducted at UCSD.

Figure 6.2 summarizes the load-displacement behavior of epoxy anchors during both uncracked and cyclic cracked concrete boundary conditions while subjected to seismic input motion of the 30% amplitude scaled reference floor motion FM02. The maximum anchor load achieved in the uncracked concrete test is approximately equal to $N_{\text{un, uncr}}$. The maximum anchor load achieved in the cracked concrete test is approximately 90% of $N_{\text{un, uncr}}$ and occurs while the crack is fully closed. The overall anchor performance between the two tests is closely comparable in terms of maximum anchor load and displacement. The reason for the similar behavior is that the North anchors experienced the maximum load while the crack was closed. It is noted that the concrete compressive strength was 4000 psi and 3500 psi for the UCSD and SDSU slab specimens respectively. Noting that concrete breakout strength is proportional to the square root of $f'_c$, (ACI 318, 2008) this could cause a difference of 7% in the resulting concrete breakout strength for the epoxy anchors.
Figure 6.2 Comparison of $N$-$\delta$ behavior for an epoxy anchor in uncracked concrete & cyclic cracked concrete, $w_{\text{max}}=0.8\text{mm}$ (Test 182 and 132: WALLE.F.N4.UC.30.1 & WALLE.F.N4.CC08.FM02.30P)

Figure 6.3 summarizes the load-displacement behavior of expansion anchors during both uncracked and cyclic cracked concrete boundary conditions, where $w_{\text{max}}=0.5\text{mm}$, due 100 and 150% amplitude scaled reference floor motion FM02. A 150% amplitude was required to fail the uncracked specimen, while only 100% scale was required for the cracked concrete case. The reader is referred to the section on “motions” for a detailed comparison between the UCSD and SDSU table motions, namely that the UCSD motions have more high frequency content above 4 Hz. The load in all four anchors exceeds the monotonic uncracked envelope for the uncracked concrete case. The South pair of anchors for the cracked concrete case experiences more than two times the displacement than the uncracked case.
Figure 6.3 Comparison of $N$-$\delta$ behavior for an expansion anchor, in uncracked concrete & cyclic cracked concrete, $w_{max}=0.5\text{mm}$ (Test 185 and 146: WALLE.F.N5.UC.150.1 & V2.WALLE.F.N5.CC05.FM02.100P)

Figure 6.4 summarizes the load-displacement behavior of expansion anchors during both uncracked and cyclic cracked concrete boundary conditions, where $w_{max}=0.8\text{mm}$, due 100 and 150% amplitude scaled reference floor motion FM02. A 150% amplitude was required to fail the uncracked specimen, while only 125% scale was required for the cracked concrete case. The reader is referred to the section on “motions” for a detailed comparison between the UCSD and SDSU table motions, namely that the UCSD motions have more high frequency content above 4 Hz. The load in all four anchors exceeds the monotonic uncracked envelope for the uncracked concrete case. The South pair of anchors for the cracked concrete case experience approximately 1” of displacement before losing all strength. The same is observed for the North pair of anchors for the uncracked concrete case.
Figure 6.4 Comparison of $N$-$\delta$ behavior for an expansion anchor, in uncracked concrete & cyclic cracked concrete, $w_{\text{max}}=0.8\text{mm}$ (Test 185 and 57: WALLE.F.N5.UC.150.1 & WALLE.F.N5.CC08.FM02.125P)

Figure 6.5 summarizes the anchor strength reduction that occurs when the anchor is located in cracked concrete during the dynamic tests. The maximum load achieved by the anchors when the cracks are open to a width of between 0.5 and 0.8 mm is 40 to 60% of the uncracked maximum load for the epoxy anchors. For the expansion anchors however; the maximum load achieved by the anchors is when the cracks are open to a width of between 0.5 and 0.8 mm is 70 to 80% of the uncracked maximum load. The $\Lambda$ factors from monotonic reference testing were approximately 50% and 70% for the epoxy and expansion anchors respectively when located in 0.8mm cracks. This suggests that the $\Lambda$ factors obtained from monotonic testing may also be appropriate for dynamic behavior (e.g. cyclic loading and cyclic cracking).
6.3 Epoxy Anchor Behavior

The epoxy anchor was tested in uncracked and cyclic cracked concrete using the FM02 reference input motion. The anchor behavior in terms of the load-displacement-crack width ($N$-$\delta$-$w$) relationship as well as failure mode are described for each test in the following sections.

6.3.1 Uncracked Concrete

The epoxy anchor experienced concrete breakout failure at an input motion scale of 30%. The South-West and South-East anchors failed by concrete breakout (Figure 6.6). The dynamic $N$-$\delta$-$w$ behavior is shown in Figure 6.7. The load-displacement behavior is very similar to the uncracked concrete monotonic reference envelope. The epoxy anchors remain very stiff until development of a breakout cone which is accompanied by a sharp and complete drop of anchor load and increase in anchor displacement. It should be noted that this test was performed at SDSU by Hoehler and Dowell (2010); however, the results are included here for comparison with the tests performed in cyclic cracked concrete.
Figure 6.6 Failure photograph, South-East anchor close-up, epoxy anchor, uncracked concrete (Test 182: WALLE.F.N4.UC.30.1), test performed at SDSU
Figure 6.7 $N-\delta-w$ results, epoxy anchor, uncracked concrete (Test 182: WALLE.F.N4.UC.30.1), test performed at SDSU
6.3.2 Cyclic Cracked Concrete \((w_{\text{max}} = 0.08 \text{ mm})\)

The epoxy anchor experienced failure at an input motion scale of 30 % in cyclically cracked concrete having a maximum crack width \((w_{\text{max}})\) of 0.8mm. The South-West and South-East anchors failed by concrete breakout (Figure 6.8).

![Figure 6.8 Failure photograph, top view, epoxy anchor, \(w_{\text{max}}=0.8\text{mm}\) (Test 132: WALLE.F.N4.CC08.FM02.30P)](image)

A close-up photograph of the anchor failure is given in Figure 6.9. The anchor exhibits a concrete breakout cone that is divided into approximately two ellipses with the crack bisecting through the center of the failure cone. It is noted that incipient cone development is observable in the North pair of anchors as well (Figure 6.8).
Anchor load-displacement-crack width behavior is summarized in Figure 6.10. The dynamic load-displacement behavior stays entirely within the uncracked monotonic envelope. The NW and NE anchors experience loads above the monotonic envelope for cracked concrete, but the excursions above the cracked envelope occur only when the crack width is very small (< 0.2mm). For the uncracked concrete case, the majority of the dynamic behavior follows the uncracked concrete monotonic envelope with a slightly higher initial stiffness for all anchors. In addition, the South pair of anchors experience a brief excursion above the monotonic envelope. It is noted that for the epoxy anchors, the dynamic behavior closely matches and follows the monotonic envelope “backbone” curves.
Figure 6.10 $N$-$\delta$-$w$ results, epoxy anchor, $w_{max}=0.8\text{mm}$ (Test 132: WALLE.F.N4.CC08.FM02.30P)
6.4 Expansion Anchor Behavior

6.4.1 Uncracked Concrete

The expansion anchor experienced failure at an input motion scale of 150%. The North-West and North-East anchors failed by pull-through (Figure 6.11). The expansion clips stayed in the drill hole. The dynamic $N$-$\delta$-$w$ behavior in uncracked concrete is shown in Figure 6.12. This test was performed at SDSU by Hoehler and Dowell (2010); however, the results are included here for comparison with the tests performed in cyclic cracked concrete.

![Failure photograph, North-West anchor close-up, expansion anchor, uncracked concrete, showing residual displacement of the anchor nut (Test 185: WALLE.F.N5.UC.150.1)](image_url)

Figure 6.11 Failure photograph, North-West anchor close-up, expansion anchor, uncracked concrete, showing residual displacement of the anchor nut (Test 185: WALLE.F.N5.UC.150.1)
Figure 6.12 N-δ-w results, expansion anchor, uncracked concrete (Test 185: WALLE.F.N5.UC.150.1)
6.4.2 Cyclic Cracked Concrete ($w_{\text{max}} = 0.05 \text{ mm}$), Version 1

The expansion anchor experienced failure at an input motion scale of 100% in cyclically cracked concrete having a maximum crack width ($w_{\text{max}}$) of 0.5mm. The South-West and South-East anchors failed by pull-through whereby the expansion clip was left in the drilled hole (Figure 6.13). The input acceleration was inadvertently stopped early, at 90% into the duration of the test due to a tripped limit on the shake table controller. For this reason, the test was repeated using a new test setup, herein denoted as “Version 2”. When comparing results of the failure test, only the Version 2 results are used in subsequent chapters.

![Failure photograph, top view, expansion anchor, $w_{\text{max}}=0.5\text{mm}$ (Test 126: WALLE.F.N5.CC05.FM02.100Pa)](image)

A close-up photograph of the anchor failure is shown in Figure 6.14. The anchor exhibited a residual displacement of approximately 1.5” as measured by the final location of the bottom of the nut above its starting position at the top of the load washer.
Anchor load-displacement-crack width behavior for the expansion anchor is summarized in Figure 6.15. In all cases, the dynamic behavior generally follows the monotonic envelop backbone curves with an increase in anchor capacity of up to 35%, likely attributable to high strain rates.
Figure 6.15 $N$-$\delta$-$w$ results, expansion anchor, $w_{\text{max}}=0.5\text{mm}$ (Test 126: WALLE.F.N5.CC05.FM02.100Pa)
6.4.3 Cyclic Cracked Concrete ($w_{\text{max}} = 0.05 \text{ mm}$), Version 2

The torque-controlled expansion anchor experienced failure at an input motion scale of 100% in cyclically cracked concrete having a maximum crack width ($w_{\text{max}}$) of 0.5mm in the Version 2 test. The South-West and South-East anchors failed by pull-through whereby the expansion clip was left in the drilled hole (Figure 6.16).

![Failure photograph, top view, expansion anchor, $w_{\text{max}}=0.5\text{mm}$ (Test 146: V2.WALLE.F.N5.CC05.FM02.100P)](image)

Figure 6.16 Failure photograph, top view, expansion anchor, $w_{\text{max}}=0.5\text{mm}$ (Test 146: V2.WALLE.F.N5.CC05.FM02.100P)

A close-up photograph of the anchor failure is shown in Figure 6.17. The anchor exhibited a residual displacement of approximately 1.7” as measured by the final location of the bottom of the nut above its starting position at the top of the load washer.
Anchor load-displacement-crack width behavior for the expansion anchor is summarized in Figure 6.18. In all cases, the dynamic behavior generally follows the monotonic envelop backbone curves with an increase in anchor capacity of up to 35%, likely attributable to high strain rates. It was observed that the dynamic load stays within the cracked concrete monotonic envelope when the cracks were cycled open.
Figure 6.18 $N$-$\delta$-$w$ results, expansion anchor, $w_{\text{max}}=0.5\text{mm}$ (Test 146: V2.WALLE.F.N5.CC05.FM02.100P)
6.4.4 Cyclic Cracked Concrete ($w_{\text{max}} = 0.08 \text{ mm}$)

The expansion anchor experienced failure at an input motion scale of 100% in cyclically cracked concrete having a maximum crack width ($w_{\text{max}}$) of 0.8mm. The South-West and South-East anchors failed by pull-through (Figure 6.19) whereby the expansion clip was left in the drilled hole (Figure 6.20). It is noted that the photographs in are taken following testing to a scale factor of 125%, failure however; which resulted in significant anchor displacement, approximately 0.5”, was identifiable during the motion prior to this, which was scaled to 100%.

Figure 6.19 Failure photograph, elevation view of WALLE, expansion anchor, $w_{\text{max}}$=0.8mm
(Test 57: WALLE.F.N5.CC08.FM02.125P)
Anchor load-displacement-crack width behavior is summarized in Figure 6.21. The dynamic behavior generally follows the monotonic envelop backbone curves. The initial stiffness of the anchors was similar to the monotonic tension tests. It was observed that the dynamic load went above the uncracked concrete monotonic envelope for the North-East anchor by approximately 35% likely attributable to strain rate effects.
Figure 6.21 N-δ-w results, expansion anchor, \(w_{max} = 0.8\text{mm}\) (Test 57: WALLE.F.N5.CC08.FM02.125P)
6.5 Drop-In Anchor Behavior

The drop-in anchor experienced failure at an input motion scale of 100% in cyclically cracked concrete having a maximum crack width ($w_{max}$) of 0.8mm. The North-West and North-East anchors failed by concrete breakout (Figure 6.22). The North-East anchor developed a full concrete breakout cone whereas the North-West anchor the breakout cone was not fully cracked through to the surface based on visual inspection; however, the anchor experienced significant displacement (> 0.5 in.) and a hollow cone beneath the surface was found by sounding the concrete with a metal hammer to determine the extent of damage to the concrete below the surface. The outline of the cone below the surface is shown with a dashed outline in the photograph.

![Failure photograph, top view, drop-in anchor, $w_{max}=0.8$mm (Test 141: WALLE.F.N7.CC08.FM02.100P)](image)
A close-up photograph of the North-East anchor failure is shown in Figure 6.23. The anchor exhibits a concrete breakout failure cone. The cone did not extend into a full circle, nor did it pullout completely from the surrounding concrete; however, the capacity of the anchor was effectively reduced to zero based on the test data.

![Figure 6.23 Failure photograph, North-East anchor close-up, drop-in anchor, w_{max}=0.8mm (Test 141: WALLE.F.N7.CC08.FM02.100P)](image)

Anchor load-displacement-crack width behavior for the drop-in anchor is summarized for cyclic cracked concrete with a maximum crack width of 0.8mm in Figure 6.24. The maximum load achieved during the dynamic testing was approximately 60% of $N_{am,uncr}$ indicating that even though the cracks were closed during portions of the test, the anchor was unable to develop its full uncracked reference strength. Likewise, when the cracks were open, the maximum load achieved was approximately 55% of $N_{am,cr}$. It was observed from the test video that the anchors experienced partial pullout and then were pushed back into the holes when the WALLE reversed direction and the SAMUs went into compression. The load-displacement curves do not indicate a well defined “failure” point for the drop-in anchors, rather a gradual loss of stiffness and strength occurred after the anchor displacement surpassed approximately 0.25 inches.
Figure 6.24 $N$-$\delta$-$w$ results, drop-in anchor, $w_{\text{max}}=0.8\text{mm}$ (Test 141: WALLE.F.N7.CC08.FM02.100P)
6.6 Undercut Anchor Behavior

6.6.1 Cyclic Cracked Concrete ($w_{\text{max}} = 0.08 \text{ mm}$), Version 1

In the version 1 failure test, the undercut anchor experienced failure at a target input motion scale of 60% in cyclically cracked concrete having a maximum crack width ($w_{\text{max}}$) of 0.8mm. The South-West and South-East anchors failed by concrete breakout (Figure 6.25). The South-East anchor developed a full concrete breakout cone, whereas at the South-West anchor, the breakout cone was not fully pulled out; however, the anchor experienced significant displacement (> 0.3 in.) and a hollow cone was found by sounding the concrete with a metal hammer to determine the extent of damage to the concrete below the surface.

![Figure 6.25 Failure photograph, top view, undercut anchor, $w_{\text{max}}=0.8\text{mm}$ (Test 51: WALLE.F.N9.CC08.FM02.60P)](image)
A close-up photograph of the anchor concrete cone failure is given in Figure 6.26. One-half of the concrete breakout cone became dislodged on one side of the crack, while most of the other side of the cone did not breakout. This sometimes happens when testing in cracked concrete as the stress field is interrupted into two “lobes” by the crack bisecting the failure cone, as described by Eligehausen et al., 2006.

![failure photograph](image)

**Figure 6.26 Failure photograph, South-West anchor close-up, undercut anchor, \( w_{\text{max}} = 0.8 \text{mm} \) (Test 51: WALLE.F.N9.CC08.FM02.60P)**

Anchor load-displacement-crack width behavior for the undercut anchor is summarized for cyclic cracked concrete with a maximum crack width of 0.8mm in Figure 6.27. The initial stiffness of the North pair of anchors closely matched the uncracked monotonic envelope. The North pair of anchors remained essentially “elastic” and undamaged. The maximum load achieved during the dynamic testing while a crack was closed was 100% of \( N_{\text{um,uncr}} \) for the NE anchor. The maximum load achieved during the dynamic testing while a crack was open to 0.8mm was approximately 130% of \( N_{\text{um,cr}} \) for the SE anchor.
Figure 6.27 N-δ-w results, undercut anchor, $w_{\text{max}}=0.8\text{mm}$ (Test 51: WALLE.F.N9.CC08.FM02.60P)
6.6.2 Cyclic Cracked Concrete ($w_{\text{max}} = 0.08 \text{ mm}$), Version 2

In the version 2 failure test, the undercut anchor experienced failure at a target input motion scale of 60% in cyclically cracked concrete having a maximum crack width ($w_{\text{max}}$) of 0.8mm. The South-East anchor failed by concrete breakout (Figure 6.28). A close-up photograph of the anchor failure is given in Figure 6.29. The failure surface of the cone initiates from the bearing surface of the undercutting blades.

Figure 6.28 Failure photograph, top view, undercut anchor, $w_{\text{max}}=0.8\text{mm}$ (Test 120: V2.WALLE.F.N9.CC08.FM02.60P)
Anchor load-displacement-crack width behavior for the undercut anchor is summarized for cyclic cracked concrete with a maximum crack width of 0.8mm in Figure 6.30. The initial stiffness of the North pair of anchors closely matched the uncracked monotonic envelope. The SW anchor experienced an offset in displacement of approximately 0.1 inches during the first load cycle where the cracks opened to greater than 0.8mm. The maximum load achieved during the dynamic testing while a crack was closed was approximately equal to $N_{um,uncr}$ for the SW anchor. The maximum load achieved during the dynamic testing while a crack was open to 0.8mm was approximately 133% of $N_{um,cr}$ for the SW anchor.
Figure 6.30 $N$-$\delta$-$w$ results, undercut anchor, $w_{\text{max}}$=0.8mm (Test 120: V2.WALLE.F.N9.CC08.FM02.60P)
6.6.3 Cyclic Cracked Concrete ($w_{max} = 0.08 \text{ mm}$), In-Phase Testing: FM14

During the in-phase failure test, the undercut anchor experienced failure at an imposed WALLE displacement of approximately 1.4” at the center of mass (flexible WALLE) in cyclically cracked concrete having a maximum crack width ($w_{max}$) of 0.8mm. The North-West anchor failed by concrete breakout (Figure 6.31).

![Figure 6.31 Failure photograph, top view, undercut anchor, $w_{max}=0.8\text{mm}$ (Test 179: STRUCT.N9.CC08.CR14.D14c)](image-url)

A close-up photograph of the anchor concrete breakout failure is given in Figure 6.32. The anchor exhibits a concrete breakout cone that is divided into approximately two ellipses with the crack bisecting through the center of the failure cone. It is noted that the actual diameter of the concrete breakout failure surface is approximately 14” on average, which is 77% larger than the theoretical value of $3h_{cf} = 7.9"$ which is used in the ACI 318 (2008) design equations. The angle
of the cone changes to a very flat angle towards the surface of the concrete. This represents “flaking” of the concrete surface, which likely does not contribute significantly to the strength of the concrete breakout cone.

Figure 6.32 Failure photograph, North-West anchor close-up, undercut anchor, \( w_{\text{max}} = 0.8\text{mm} \) (Test 179: STRUCT.N9.CC08.CR14.D14c)

Anchor load-displacement-crack width behavior for the undercut anchor is summarized for cyclic cracked concrete with a maximum crack width of 0.8mm in Figure 6.33. The NW anchor was the one anchor that failed during the test due to concrete breakout. This anchor cycled along the initial stiffness loading curve (Figure 6.33), then a concrete breakout cone formed as the anchor reached approximately 9.5 kips, or 85% of \( N_{\text{um,uncr}} \). The maximum load achieved during the dynamic testing, while a crack was closed was 157% of \( N_{\text{um,cr}} \) for the NW anchor. The maximum load achieved during the dynamic testing, while a crack was open to 0.8mm was approximately 95% of \( N_{\text{um,cr}} \) for the SE anchor.
Figure 6.33 \(N-\delta-w\) results, undercut anchor, \(w_{max}=0.8\text{mm}\) (Test 179: STRUCT_N9_CC08_CR14_D14c)
6.6.4 Cyclic Cracked Concrete ($w_{max} = 0.08$ mm), In-Phase Testing: CR2Hz, D1Hz

In-phase test STRUCT.N9.CC08.CP2.D1c was not technically a “failure test” because the anchor was not loaded to failure; however, the results are presented here because the anchor loading amplitude was large enough to have the data considered as part of the failure test series. The test protocol was described in detail in Chapter 4. The resulting anchor load-displacement-crack width behavior for the undercut anchor is summarized for cyclic cracked concrete with a maximum crack width of 0.8mm in Figure 6.34. The maximum load achieved during the dynamic testing, while a crack was open to 0.8mm was approximately 90% of $N_{um,cr}$ for the SE anchor. This test also demonstrates the uneven load distribution between a pair of anchors on the same side of WALLE (e.g. NW versus NE anchor where the maximum load was approximately 90% of $N_{um,cr}$ and 25% of $N_{um,cr}$ respectively).
Figure 6.34 \( N-\delta-w \) results, undercut anchor, \( w_{\text{max}}=0.8\,\text{mm} \) (Test 176: STRUCT.N9.CC08.CP2.D1c)

**Figure Details:**
- **STRUCT_N9_CC08_CP2_D1c**
- Anchor Type: Undercut

**Graphs:**
1. **Crack Width vs. Time:**
   - NorthWest
   - NorthEast
   - SouthWest
   - SouthEast
   - Monotonic Uncracked
   - Monotonic Cracked

2. **Anchor Load vs. Time:**
   - NorthWest
   - NorthEast
   - SouthWest
   - SouthEast

3. **Anchor Disp vs. Time:**
   - NorthWest
   - NorthEast
   - SouthWest
   - SouthEast

4. **Crack Width vs. Anchor Load:**
   - NorthWest
   - NorthEast
   - SouthWest
   - SouthEast

**Key Values:**
- Max NW = 0.85 mm
- Max NE = 0.93 mm
- Max SW = 0.82 mm
- Max SE = 0.86 mm
- Max NW = 4.21 kip
- Max NE = 1.26 kip
- Max SW = 2.56 kip
- Max SE = 3.67 kip
- Max NW = 0.12 in
- Max NE = 0.16 in
- Max SW = 0.03 in
- Max SE = 0.04 in
6.7 Anchor Strain Rate

Strain rate can play an important role in anchor response and can lead to ultimate strength increases on the average of 20% for earthquake relevant strain rates (Hoehler et. al, 2011). Earthquake relevant strain rate rise times were defined as being in the range of a 0.025 and 0.25 seconds (e.g Hoehler et al., 2011). For an anchored NCS, anchor loading rate should be dependent on the fundamental natural period of the NCS, as this is driving the anchor loading. That is, if the NCS responds to seismic input primarily in its first mode. Thus, the theoretical rise time for anchor loading may be approximated as one quarter of one cycle of vibration of the NCS (i.e. rise time $\cong T_1/4$), where $T_1$ is the first mode period of the NCS (Hoehler et al., 2011). For comparison of theoretical values with experimental observation, the axial tension strain rates were calculated from the WALLE failure tests for anchor loads above 50% $N_{num,cr}$. This load level was selected as the anchor is approaching failure at large tension loads, but is still low enough to provide a diverse data set. The observed strain rates were on average 0.09 seconds and ranged from 0.04 seconds to 0.16 seconds. The fundamental period of WALLE at incipient anchor failure was approximately 0.37 seconds as discussed in the chapter on system characterization thus rise time may be approximated as $T_{NCS}/4 = T_{WALLE}/4 = 0.093$ seconds which corresponds well to the observed average of 0.09 seconds.

6.8 Anchor Load Variability

An important result from these experiments was that it was observed that anchor load was not equally shared between a pair of anchors on the North or South side of WALLE even though seismic input was uniaxial in the North-South direction. In design practice it is typically assumed that anchor forces may be proportioned evenly assuming a rigid baseplate. This can be unconservative if one anchor carries more load without the load redistribution typically assumed.
The following analysis determines the variability in anchor load sharing, herein termed load variability. It was observed that the anchor load sharing is very uneven at low loads and tends to somewhat even out at larger loads. Therefore to examine this anchor load sharing was determined applying several load cutoff values. The loads considered were the maximum load per load cycle, $N_{\text{max},k}$ on an anchor and the corresponding load on the other anchor of the pair at the same instant in time, $N_{i,k}$. The load variability is then determined for each anchor as follows:

$$N_{\text{var}} = \frac{N_{\text{max},k}}{N_{\text{max},k} + N_{i,k}} \quad 6-1$$

Where $N_{\text{var}}$ is the percentage of load carried by the more highly loaded anchor, ranging from 0 to 2. Note when $N_{\text{var}} = 0.5$, the anchors are sharing load evenly. When $N_{\text{var}} = 1$ it indicates that the anchor under consideration is carrying 100% of the load and the other anchor has zero load. When $N_{\text{var}} \sim 0$ it indicates that the anchor under consideration is carrying the a very small load and the other anchor has is carrying most of the load in comparison. Figure 6.35 displays a histogram of the load variability, $N_{\text{var}}$ for a load cutoff of 5%$N_{\text{um,cr}}$ applied. It is observed that a large amount of data is at 0.5 which indicates that in many cases the load is shared 50/50 between a pair of anchors; however, there is also a cluster of data around 0.98 which indicates that for those load cycles, one anchor is carrying almost all of the load. The coefficient of variation, CV, is 0.36 for a load cutoff of 5%$N_{\text{um,cr}}$. 
Design practice is focused on maximum anchor response; therefore it is also useful to examine anchor load variability at larger anchor loads. Figure 6.36 displays a histogram of the load variability, $N_{var}$, for a load cutoff of 25% $N_{um,cr}$ applied. It is observed that a large amount of data is at 0.5 which indicates that in many cases the load is shared 50/50 between a pair of anchors; however, there is also a smaller cluster of data around 0.96 which indicates an uneven distribution of load where one anchor is carrying almost all of the load. The coefficient of variation, $CV$, drops to 0.29 for a load cutoff of 25% $N_{um,cr}$. Figure 6.37 summarizes the $CV$ for load cutoffs of between 1 and 25% $N_{um,cr}$. It can be seen that the variability is very large for 1% load cutoff but tends to stabilize as the load cutoff increases, approaching a $CV$ of approximately 30% for large anchor loads. If one wishes to account for the variability in anchor load distribution, it is recommended to use a $CV$ equal to 30%.
Figure 6.36 Anchor load sharing (load cutoff of 25% \(N_{um,cr} \text{ applied} \))

Figure 6.37 Load sharing four anchors (load cutoff of 5% \(N_{um,cr} \text{ applied} \))
6.9 Anchor Shear Load

The WALLE tests were primarily designed to be tension dominated; however, a portion of the base shear is resisted by the anchors and a portion through fiction between the SAMUs and the concrete as well as the toe of the SAMUs digging in to the concrete as WALLE rocks about the toe. This section examines the anchor shear measurements and it is observed that anchor shear was generally low and can be neglected from interaction with the tension capacity of the anchors. Figure 6.38 summarizes the anchor shear load history for a typical failure test on the undercut anchors using the reference floor motion FM02 at 60% amplitude scale. It is noted that the anchors do not evenly carry the shear load as typically assumed in design practice. It is also noted that due to the WALLE aspect ratio, the maximum anchor shear load, $V_u$, 1.06 kips, is small compared to its allowable ultimate shear capacity, $\phi V_n$, approximately 6 kips. The demand to capacity ratio in shear, $V_u/\phi V_n$ is 18%. Some design codes stipulate that tension-shear interaction can be ignored for shear less than 20% of the anchor shear strength (e.g. ACI 318, 2008). The WALLE base shear ($V_{WALLE}$) is calculated from the WALLE mass times acceleration at the center of mass (CM) of WALLE. The portion of the base shear carried by the anchors ($V_{anchors}$), is also shown. It is noted that at the beginning of the test, the majority of the base shear is resisted through friction between the SAMUs and the concrete; however, as the friction is overcome the anchors carry a larger portion of the base shear.
Table 6.2 summarizes the maximum anchor shear load and base shear for the failure tests. It may be generalized that the maximum anchor shear loads during these tests are small compared with the shear capacity of the anchors, on average 15% and a maximum of 31% during one test. Therefore it is concluded that the interaction between shear and tension loading may be ignored in the analysis of the test results.
Table 6.2 Anchor shear summary

<table>
<thead>
<tr>
<th>Anchor</th>
<th>Test Name</th>
<th>Max. Base Shear (kip)</th>
<th>Max. Total Anchor Shear (kip)</th>
<th>Max. Anchor Shear (kip)</th>
<th>$V_{\text{max}}/\phi V_n$</th>
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<tbody>
<tr>
<td>Epoxy</td>
<td>WALLE.F.N4.UC.30.1</td>
<td>4.42</td>
<td>1.00</td>
<td>0.48</td>
<td>5%</td>
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<td></td>
<td>WALLE.F.N4.CC08.FM02.30P</td>
<td>2.50</td>
<td>2.12</td>
<td>1.54</td>
<td>16%</td>
</tr>
<tr>
<td>Expansion</td>
<td>WALLE.F.N5.UC.150.1</td>
<td>8.04</td>
<td>3.25</td>
<td>1.27</td>
<td>14%</td>
</tr>
<tr>
<td></td>
<td>WALLE.F.N5.CC05.FM02.100Pa</td>
<td>4.76</td>
<td>0.86*</td>
<td>0.47</td>
<td>5%</td>
</tr>
<tr>
<td></td>
<td>V2.WALLE.F.N5.CC05.FM02.100P</td>
<td>5.31</td>
<td>1.70</td>
<td>1.03</td>
<td>11%</td>
</tr>
<tr>
<td></td>
<td>WALLE.F.N5.CC08.FM02.125P</td>
<td>5.23</td>
<td>2.18</td>
<td>1.46</td>
<td>16%</td>
</tr>
<tr>
<td>Drop-In</td>
<td>WALLE.F.N7.CC08.FM02.100P</td>
<td>1.90</td>
<td>2.82</td>
<td>1.84</td>
<td>20%</td>
</tr>
<tr>
<td>Undercut</td>
<td>WALLE.F.N9.CC08.FM02.60P</td>
<td>4.91</td>
<td>3.16</td>
<td>1.06</td>
<td>18%</td>
</tr>
<tr>
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<td>V2.WALLE.F.N9.CC08.FM02.60P</td>
<td>4.28</td>
<td>1.89</td>
<td>1.87</td>
<td>31%</td>
</tr>
<tr>
<td>AVERAGE</td>
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<td>4.59</td>
<td>2.11</td>
<td>1.22</td>
<td>15%</td>
</tr>
<tr>
<td>MAXIMUM</td>
<td></td>
<td>8.04</td>
<td>3.25</td>
<td>1.87</td>
<td>31%</td>
</tr>
</tbody>
</table>

*One SAMU pin was faulty; therefore measurements are derived from the remaining 3 SAMUs

6.10 Summary Remarks

Based on experimental observations and measurements related to the response of the anchored WALLE, the following conclusions can be made:

- The maximum anchor tension load was bounded by -20% to +20% of $N_{\text{uncr}}$ when an anchored NCS was subjected to inertial earthquake loading in uncracked concrete. This is within the scatter range of typical monotonic test results. The maximum anchor load was approximately equal to $N_{\text{uncr}}$ which suggests that for inertial loading in uncracked concrete, the use of the mean uncracked capacity from monotonic reference tests is reasonable to predict dynamic anchor demands at failure.

- The maximum anchor tension load was bounded by -20% to +80% of $N_{\text{cr}}$ when an anchored NCS was subjected to inertial earthquake loading and crack cycling, for anchors that have a seismic approval for use in cracked concrete (epoxy, expansion, and undercut anchors). The lower bound is within the range of normal scatter of anchor test data and the upper bound shows an increase in capacity which in most cases was due to
the cyclic cracks being closed or partially closed when the maximum anchor load occurred.

- The one anchor that did not have a seismic approval for use in cracked concrete (drop-in anchor) observed a maximum anchor load bounded by -45% to +40% of $N_{um,cr}$. This result suggests that this anchor can perform worse in cyclic cracks than anticipated from monotonic reference testing.

- There were cases where the maximum anchor load exceeded the uncracked capacity from monotonic reference tests. While some of this increase is anticipated to be due to natural variability, it is hypothesized that this increase in anchor strength may be due to strain rate effects as well as compaction of the concrete around the anchor bearing surfaces.

- The $N$-$\delta$ relationship under dynamic inertial loading and crack cycling is generally bounded by the monotonic reference test envelope for both load and displacement for all anchor types tested.

- Typically 1 to 2 anchors failed of the 4 anchor arrangement.

- Anchor axial strain rate load rise time may be approximated by $T_{NCS}/4$ for anchored NCSs.

- Test videos were produced with anchor data plots that allow for the synthesis of results. The videos are the first time, to the author’s knowledge, that anchor failure in cyclically cracked concrete due to inertial seismic loading has been captured on video with corresponding instrumentation of input acceleration, input displacement, NCS response acceleration, NCS response displacement, crack width, anchor load, and anchor displacement.

- The coefficient of variation on anchor load sharing between a pair of anchors on one side of WALLE was 0.3 when considering a load cutoff of 25% $N_{um,cr}$. 
• The maximum shear load on the anchors was low and the interaction of shear and tension loading can be neglected in the analysis of the experimental data set.
Chapter 7 Pre-Failure and Failure Test Series: Response Analysis

7.1 Introduction

The goal of this chapter is to compare the local anchor behavior in terms of load and displacement as well as system response in terms of WALLE acceleration and displacement response for the various anchor types tested. Observations are made regarding the effect of the anchor behavior in cyclic cracks on system performance. Comparisons are made between anchor behavior of epoxy, expansion, drop-in, and undercut anchors in cyclic cracks under dynamic inertial loading and it is suggested that the monotonic tension reference curves in uncracked and cracked concrete may be sufficient to predict anchor behavior and system response during inertial seismic loading coupled with cyclic cracking. This hypothesis however must be further examined by predictive modeling and comparison with experimental results.

7.2 Verification of Anchor Forces

Before interpreting system results and drawing comparisons between different anchor types, a system level analysis was performed on the anchor forces to determine the reliability of the load washers as mounted in the SAMUs. This system force check is in addition to the static calibration that was already performed as part of normal instrumentation calibration. An incremental equivalent static check was performed at each instant in time based on the dynamic and kinematics of the system. The equivalent static check methodology and results are discussed below. Predicted anchor forces are compared with experimentally obtained anchor forces. From these analyses it is concluded that the load washers are providing reliable load measurements. Additionally, a response spectrum analysis was conducted using the floor spectrum and the period
of the WALLE measured from the WALLE response history. The predicted anchor forces from
the response spectrum analysis are compared with experimentally obtained anchor forces and
found to be on average in generally good agreement. The experiments however indicate a large
variability in distribution of anchor forces, suggesting that the anchors do not share equally the
imposed forces, even when the pair of anchors is located on the same side of WALLE where in
design practice, the load would be divided 50/50 between the two anchors. Variations on the
order of +/- 50% were observed between predicted and achieved anchor forces.

7.2.1 Equivalent static analysis procedure

An equivalent static analysis of anchor forces was conducted to assure that the load
washers were working properly. Starting with the equation of motion for a SDOF system
subjected to base excitation, the following equations govern the motion:

\[ u_i = u + u_g \]  \hspace{1cm} 7-1

\[ \ddot{u}_i = \ddot{u} + \ddot{u}_g \]  \hspace{1cm} 7-2

\[ m\ddot{u} + c\dot{u} + ku = -m\ddot{u}_g \]  \hspace{1cm} 7-3

\[ m\ddot{u}_i + c\dot{u}_i + ku_i = 0 \]  \hspace{1cm} 7-4

Where \( \ddot{u}_i \) is the absolute, total, acceleration of the mass, \( \ddot{u} \) is the acceleration of the
mass relative to the ground, \( \ddot{u}_g \) is the acceleration of the ground, \( u_i \) is the absolute, total,
displacement of the mass, \( u \) is the displacement of the mass relative to the ground, \( u_g \) is the
displacement of the ground and \( \dot{u} \) is the velocity of the mass relative to the ground. The
parameters \( m, k \) and \( c \) represent the mass, stiffness and damping of the system, respectively.
Using the concept of equivalent static force $f_s$, the forces in the anchors can be computed at each instant of time using the following equation.

$$f_s(t) = ku(t) = m\ddot{u}_i(t) + c\dot{u}_i(t)$$  \hspace{1cm} 7-5

Depending on the motion amplitude during the tests, WALLE can respond in a non-linear manner, therefore the stiffness is not directly known at each instant of time. To calculate anchor forces, one must use the measured relative displacement or absolute acceleration and integrated velocity obtained during the test (i.e. $f_s(t) = m\ddot{u}_i(t) + c\dot{u}(t)$). At peak displacement, the velocity is zero, thus damping has a negligible effect on peak anchor forces. To determine the relative effect of damping, the following equations are evaluated assuming that damping is approximately 2% of critical and the fundamental frequency of WALLE is 4Hz. Considering these parameters, it was determined that the maximum contribution of damping to the system acceleration was 0.1 $g$ at the maximum velocity achieved during the tests. The maximum anchor forces occur when system displacement is at a maximum, therefore, since velocity and displacement are out of phase, and the amount of equivalent viscous damping is small, damping forces are assumed to be negligible and are ignored in the subsequent analysis.

$$c = 2m\zeta\omega_n$$  \hspace{1cm} 7-6

$$\omega_n = 2\pi f_n = 2\pi (4\text{Hz}) = 25.13 \frac{\text{rad}}{\text{s}}$$  \hspace{1cm} 7-7

$$c = 2m\zeta\omega_n = 2m(0.02) \left( 25.13 \frac{\text{rad}}{\text{s}} \right) = 1.01m$$  \hspace{1cm} 7-8

$$f_a = m\ddot{u}_i$$  \hspace{1cm} 7-9
\[ f_d = c\ddot{u} = 1.01 \frac{lb}{s} \left( \frac{W}{386.2 \frac{in}{s^2}} \right) \left( 41 \frac{in}{s} \right) = 0.1 W \]

Where \( f_a \) and \( f_d \) are the equivalent static forces due to acceleration and damping respectively, \( W \) is the weight of the system, \( \zeta \) is the damping expressed as a percentage of critical damping, and \( \omega_n \) is the circular natural undamped frequency of vibration. The acceleration of WALLE in the direction of shaking at the center of mass \((a_{cm})\) and the displacement of WALLE in the direction of shaking at the center of mass \((d_{cm})\) were not directly recorded during the tests because the weight plates interfered with placing instrumentation directly at the center of mass. Therefore, measurements of acceleration and displacement were made at locations near the center of mass and were translated to the center of mass by the kinematic equations as described in the section on data post-processing. Figure 7.1 depicts a free body diagram of the forces acting on WALLE when displaced to the North \((+d_{cm})\).
Figure 7.1 Free body diagram of flexible WALLE showing: (a) the assumed displaced shape of WALLE, (b) forces due to self weight in the displaced position, and (c) forces due to seismic inertial loading.

From the geometry of WALLE and the SAMUs, $a=2.5''$ and $b=6.5''$. From summation of the vertical forces we obtain the following equation:

$$C_{sd} + C_{nd} + T_{nd} - T_{sd} = mg = W$$ 7-11

Where $T_{sd}$ is the total tension force on the South pair of anchors due to WALLE self weight in the displaced position, $T_{nd}$ is the total tension force on the North pair of anchors due to WALLE self weight in the displaced position, $C_{sd}$ is the total compression force between the
South pair of SAMUs and the supporting concrete and $C_{nd}$ is the total compression force between the North pair of SAMUs and the supporting concrete.

The reactions against the concrete due to WALLE self weight, $C_{nd}$ and $C_{sd}$ are assumed to be equal and support 50% of the mass of WALLE, then from summation of vertical forces we obtain

$$T_{sd} = T_{nd}$$ \hspace{1cm} 7-12

From summation of moments about the centerline of WALLE, we obtain

$$T_{sd}b + T_{nd}b = mgd_{cm} = wd_{cm}$$ \hspace{1cm} 7-13

By substitution, we obtain

$$T_{sd} = \frac{wd_{cm}}{2b}$$ \hspace{1cm} 7-14

From summation of the horizontal forces we obtain the following equation:

$$V_s + V_n + V_c = ma_{cm} + cv_{cm}$$ \hspace{1cm} 7-15

where $V_s$ is the total shear force on the South pair of anchors, $V_n$ is the total shear force on the North pair of anchors and $V_c$ is the total shear friction force at the rocking “toe”, $ma_{cm}$ is the inertial force due to acceleration of the mass and $cv_{cm}$ is the velocity dependent damping force at the center of mass.

From experimental observation of SAMU pin forces, the majority of the shear is resisted at the compression toe, “digging” in to the concrete. Therefore, for this simplified model we can assume that the entire system shear is resisted by $V_c$. 
Considering anchorage loads generated by lateral forces acting on the WALLE (Figure 7.1c) and summation of moments about the compression toe $C_{na}$ we obtain the following equation when WALLE is displaced to the North ($+d_{cm}$):

$$-ma_{cm}h_{cm} - cv_{cm}h_{cm} + T_{sa}^+ (a + 2b) + T_{na}^+ a = 0$$  

where $T_{sa}^+$ is the total force on the South pair of anchors and $T_{na}^+$ is the total force on the North pair of anchors.

By assuming that the WALLE base is rigid and plane sections remain plane we obtain the following compatibility equation:

$$T_{ns}^+ + T_{sa}^+ \left( \frac{a}{a + 2b} \right) \geq 0$$  

The anchors cannot resist compression therefore the anchor tension force must be greater than or equal to zero.

By combining the previous two equations, substituting in for $T_{n}^+$ and rearranging we are able to solve for the sum of the tension force in the south anchors by the following equation:

$$T_{sa}^+ = \frac{ma_{cm}h_{cm} + cv_{cm}h_{cm}}{a + 2b + \frac{a^2}{a + 2b}} \geq 0$$  

By following the same procedure for when WALLE is displaced to the South ($-d_{cm}$) we obtain the following equations.

$$-ma_{cm}h_{cm} - cv_{cm}h_{cm} - T_{sa}^- a - T_{na}^- (a + 2b) = 0$$
By solving the previous system of equations at each instant in time during a test, we are able to compute the theoretical anchor force history based on observed measurements of WALLE acceleration and displacement for experimentation.

### 7.2.2 Equivalent static analysis average results

The results of the predicted and achieved anchor forces are presented below for select tests. The following tests were selected as representative samples (a) Figure 7.2, test 82, variable phase correlation, Stiff WALLE configuration, using undercut anchors where the maximum anchor force of the average per pair of anchors reached approximately 65%$N_{\text{um,cr}}$, or 36%$N_{\text{um,uncr}}$, (b) Figure 7.3, test 120, failure test, Flexible WALLE configuration, using undercut anchors where the maximum anchor force of the average per pair of anchors reached approximately 132%$N_{\text{um,cr}}$, or 72%$N_{\text{um,uncr}}$, (c) Figure 7.4, test 124, failure test, Flexible WALLE configuration, using expansion anchors where the maximum anchor force of the average per pair of anchors reached approximately 55%$N_{\text{um,cr}}$, or 40%$N_{\text{um,uncr}}$. $N_m$ is the mean load on a pair of anchor on the North or South side of WALLE.

\[
T_{na} = \frac{-ma_{cm} h_{cm} - cv_{cm} h_{cm}}{a + 2b + \frac{a^2}{a + 2b}} \geq 0 \quad 7-20
\]

\[
T_{sa} = T_{na} \left( \frac{a}{a + 2b} \right) \geq 0 \quad 7-21
\]
Figure 7.2 Predicted versus achieved anchor forces (Test 82: V2.WALLE.R.N9.CC08.FM08)

Figure 7.2 displays a close match between the average predicted and average experimental anchor forces, with the experimental forces being slightly larger. Figure 7.3 also displays a close match between the average predicted and average experimental anchor forces, however, the experimental forces are slightly smaller. It is noted that for the south pair of anchors in Figure 7.3, a discrepancy is observed for the low amplitude forces occurring after strong shaking, between 10 and 16 seconds. Namely, the model predicts anchor forces, while none are observed during the test. It is hypothesized that this is due to the slight loosening of the nut that occurs during the test due to anchor displacement. This relaxation of the anchors causes less transmission of inertial loads to the anchors at low amplitude shaking and more to the restoring force of the self weight of the WALLE, causing compression between the SAMUs and the concrete slab.
The model is not able to capture the anchor preload that is present during the initial part of the experiment (Figure 7.4) from 2 to 5.5 seconds; however, the preload is reduced during the initial load cycles occurring between 3.5 and 4.5 seconds and the preload is entirely gone after the strong pulse that occurs at approximately 6.2 seconds. After the preload is gone, the model predicts the experimental results well on average.
7.2.3 Equivalent static analysis results per anchor

A simple static analysis predicts that the resisting load on a pair of anchors on the North or South side of WALLE should be shared 50/50; however, it was observed during testing that the load sharing between a pair of anchors was not equal (Figure 7.5). The load distribution between anchors is highly variable at low to moderate loading amplitudes (upper plots). The load distribution between two anchors on a side tends to even out at high loads (lower plots). The coefficient of variation of maximum anchor force was 50% considering all failure tests series of increasing motion amplitude up to and including the final failure tests. The coefficient of variation was 35% considering only the final failure tests. The following analyses compares the maximum predicted anchor load with maximum load achieved by each individual anchor. The results for all WALLE dynamic tests (failure series and variable phase correlation series) are summarized in Figure 7.6. The model predicts the experimental anchor loads reasonably well on average throughout the entire range of anchor load amplitude.
Figure 7.5 Distribution of maximum forces in a group of four anchors under single axis seismic input – experimental results.

To further examine the model, the results are divided into failure tests and variable phase correlation tests in Figure 7.7. The model predicts test results from both test series equally. Figure 7.8 plots the results from the variable phase correlation tests divided out by flexible and stiff WALLE results. It appears that the model slightly overpredicts anchor forces for the flexible WALLE and slightly underpredicts anchor forces for the stiff WALLE. This could be due to inaccuracy in the calculation of the height to the center of mass for each WALLE configuration. For example, the SAMUs and base tube are included in the center of mass calculation but may not participate in the dynamic mass since it is directly attached to the slab.
Figure 7.6 Predicted versus achieved maximum anchor forces from all tests

Figure 7.7 Predicted versus achieved peak anchor forces (a) failure and (b) correlation tests

Figure 7.8 Predicted versus achieved peak anchor forces from correlation tests (a) flexible WALLE and (b) stiff WALLE
7.2.4 Equivalent static analysis average results per anchor pair

If an even distribution of loading between a pair of anchors is assumed, the maximum predicted loads may be compared with the average of the maximum measured loads per pair of anchors. This comparison eliminates the variability due to uneven sharing of anchor load on a pair of anchor by comparing the average with the predicted. Figure 7.9 summarizes the experimental maximum average load on a pair of anchors from the load washers from all tests with the maximum average load predicted from the static analysis using measured WALLE accelerations and displacements. To further examine the model, the results are divided into failure tests and variable phase correlation tests in Figure 7.10. The model predicts test results from both test series equally. Figure 7.11 plots the results from the variable phase correlation tests separated out by flexible and stiff WALLE results. It appears that the model slightly overpredicts anchor forces for the flexible WALLE and slightly underpredicts anchor forces for the stiff WALLE. This could be due to inaccuracy in the calculation of the height to the center of mass for each WALLE configuration. For example, the SAMUs and base tube are included in the center of mass calculation but may not participate in the dynamic mass since it is directly attached to the slab.
Figure 7.9 Predicted versus achieved mean per anchor pair max. anchor forces. all tests

Figure 7.10 Predicted versus achieved mean per anchor pair maximum anchor forces (a) failure and (b) correlation tests

Figure 7.11 Predicted versus achieved mean per anchor pair maximum anchor forces from correlation tests (a) flexible WALLE and (b) stiff WALLE
7.2.5 Floor response spectrum analysis

The previous analysis was conducted using the experimentally observed WALLE acceleration and displacement response. However, it is useful to know if the anchor loads can be accurately predicted, if only the floor acceleration history is known. Therefore, a floor response spectrum analysis was conducted to compare experimentally obtained anchor forces with analytically derived anchor forces using the measured floor response spectrum at the base of WALLE. Figure 7.12 describes a free body diagram of the WALLE forces and response spectrum analysis approach. This is the type of analysis that may be performed on a design project, when time history response analysis is performed and the response spectrum at each floor level is developed. Herein, floor spectra were calculated from the experimentally measured input motion at the base of WALLE. Two percent was assured, based on the average WALLE damping of 1.8% from previously described system identification studies. WALLE was treated as an SDOF having a natural period of vibration ($T_{WALLE}$) equal to the period determined from each test by using the frequency response function (FRF) obtained from the ratio of the power spectral density (PSD) of the WALLE response acceleration divided by the PSD of the input acceleration at the slab (base of WALLE). The result of the analysis is the maximum predicted tension $T$ in each anchor on the tension side.
The floor response spectra analysis results are summarized in Figure 7.13 for the WALLE failure series and variable phase correlation series. On average the model predicts the maximum experimentally determined anchor loads, however, there is considerable scatter in the results (±/− 50%). This variability is an issue that may need to be considered further in anchor design.
Figure 7.13 Comparison of anchor load as predicted from floor spectra analysis approach versus measured experimentally: correlation tests on stiff and flexible WALLE

The results were also categorized by stiff WALLE correlation, flexible WALLE correlation and failure test results (Figure 7.14) for the maximum average load in a pair of anchors. The response spectrum model predicts the average anchor load within a range of approximately +/- 50%. The experimentally measured force has a slight trend of being larger than predicted at load amplitudes greater than 6 kips. Considering only the failure tests, the experimental anchor load is almost always larger than the predicted load. The flexible WALLE correlation test anchor load results are the best predicted by the response spectrum model.
Figure 7.14 Comparison of average anchor load per pair of anchors on the North and South sides as predicted from floor spectra analysis approach versus measured experimentally: correlation tests and failure tests

7.3 Anchor Performance

Failure test series were conducted at increasing input motion intensity unit failure occurred for the anchor type under consideration. The following sections are provided to compare the relative anchor performance for the epoxy, expansion, drop-in, and undercut anchors. Performance is characterized by anchor load-displacement \((N-\delta)\) and load-crack width \((N-w)\) behavior, and within this section more specifically considering the peak load and displacement response.

7.3.1 Load Behavior

Anchor load capacity is the main anchor performance parameter of interest in current design methodology (ACI 318, 2008). As shown previously, anchor load capacity is reduced
when the anchor is located in a crack. Figure 7.15 summarizes the load versus crack width \((N-w)\) relationship for various anchor types during the failure test series. A failure test “series” consisted of several tests on the same anchored WALLE setup with incrementally increasing amplitude scale of the input motion until anchor failure was achieved. To compare data from different anchor types, the anchor load has been normalized by the monotonic reference test capacity in a static crack width of 0.8 mm, \(N_{um,cr}\). The input amplitude scale at which failure occurred is noted on the right y-axis. Data is shown for all load points sampled at 200 Hz \((N_i)\) and well as for the maximum load per load cycle \((N_{max,k})\). These values have been normalized by \(N_{um,cr}\) therefore are noted with an asterisk (i.e. \(N^*_i\) and \(N^*_{max,k}\)). The monotonic reference test envelope is shown as a dashed line. Although these reference values are shown linearly connected, from \(w = 0\) to 0.8mm, it is well known that the N-w relationship is not linear, but rather follows a parabolic shape with a sharp decrease in load at lower crack widths (Eligehausen and Balogh, 1995). The data does not display a direct correlation between anchor load capacity \((N_{um,cr})\) and amplitude scale at failure for the floor mounted WALLE. The drop-in anchor has the lowest strength, but had the second highest amplitude scale at failure (100%). Additionally, the anchor load capacity reduction in cracked concrete \((\Lambda)\) did not play a significant effect on amplitude scale at failure. Rather, the failure mechanism type had the greatest effect on amplitude scale at failure, with the concrete cone failures having the lowest scale 30-60% and the pull-through and pullout followed by concrete cone failure mechanisms resisting the largest scale at failure (100-125%). Figure 7.16 replots the same data set, but this time the anchor load is normalized by the more traditional normalization scheme using the monotonic reference test ultimate load in uncracked concrete \(N_{um,uncr}\). Load capacity reduction factors \((\Lambda)\) are consistent with the trends from monotonic reference tests (e.g. \(\Lambda = 0.5, 0.7, 0.4, \) and 0.8 for the epoxy, expansion, drop-in, and undercut anchors respectively).
Figure 7.15 percentage $N_{um,cr}$ vs $w$ results for failure test series at increasing amplitude scale, $w_{max,target} = 0.8$ mm, dashed line denotes monotonic test results
Figure 7.16 percentage $N_{um,uncr}$ - $w$ results for failure test series at increasing amplitude scale,

$w_{max,target} = 0.8$ mm, dashed line denotes monotonic test results
Data is extracted from only the final failure tests, where anchors actually failed, in order to examine the load versus crack width relationship at anchor failure. Figure 7.17 summarizes the \(N_{\text{cr}}/N_{\text{cr,0}}\) results for all failure tests as well as the in-phase correlation tests conducted to failure. In general the results follow the monotonic envelope well with the following exceptions. The drop-in anchor underperformed the monotonic envelope from reference tests. As a note, the drop-in anchor is the only anchor that does not have a seismic approval in cracked concrete. The expansion and undercut anchors slightly outperform the monotonic envelop. The in-phase correlation tests, SUM.STRUCT.FM14 and SUM.STRUCT.1HZ purposely targeted large anchor loads coincident with large crack widths therefore there is little load data at small crack widths; however, the dynamic test results closely match the monotonic envelope at larger crack widths. These data strongly support the current code philosophy of design based on an anchor load capacity based on the monotonic capacity in cracked concrete with an increase in capacity allowed if it can be demonstrated that the anchor is located in uncracked concrete.
Figure 7.17 percentage $N_{um,cr}$-w results for final failure tests, $w_{max, target} = 0.8$ mm, dashed line denotes monotonic test results.
The maximum achieved anchor loads by anchor type are summarized in Figure 7.18. The maximum loads are compared with the monotonic mean capacities in uncracked \((N_{\text{um,uncr}})\) and cracked concrete \((N_{\text{um,cr}})\). The crack width at maximum anchor load is also noted. The maximum anchor load for all anchor types was less than \(N_{\text{um,uncr}}\) except for the epoxy and expansion anchors when tests were conducted in uncracked concrete, which experienced an increase in strength of 8 to 19% respectively. The increase in strength during the dynamic tests is believed to occur due to strain rate effects and compaction of the concrete at the bearing surfaces. The maximum achieved anchor load amplitude was generally between \(N_{\text{um,cr}}\) and \(N_{\text{um,uncr}}\) for the tests conducted in cracked cyclic concrete except for the drop-in anchor for which the two anchors that failed, failed at approximately 55% to 60% of \(N_{\text{um,cr}}\). If the failed anchors are selected for closer inspection, it is noted that the maximum achieved load is on average approximately equal to \(N_{\text{um,cr}}\) considering all anchor types except the drop-in anchor. This leads to the observation that for anchors that are qualified for use in cracked concrete in accordance with standard test methods using \(N_{\text{um,cr}}\) for design in cyclic cracks is reasonable. However, for anchors that perform poorly, such as the drop-in anchor, designing for \(N_{\text{um,cr}}\) may be unconservative. To distinguish these poorly performing anchors, another test method in addition to the monotonic tension reference test may be required, such as a load cycling test with constant crack width or crack cycling test with constant load.
Table 7.1 summarizes the maximum anchor loads and corresponding crack width at maximum anchor load \((w @ N_{max})\) (i.e. the data used to generate Figure 7.18). The monotonic ultimate mean loads are included for reference. The failed anchors are indicated by bold text with shading. The ratio of the maximum load achieved in cyclically cracked concrete to corresponding monotonic mean ultimate reference strength is also listed \((N_{max} / N_{um,ref})\). Where \(N_{um,ref}\) is either \(N_{um,uncr}\) (if the dynamic test was conducted in uncracked concrete) or \(N_{um,cl}\) (if the dynamic test was conducted in cracked concrete). This ratio provides a general indication of the loads in cyclic cracked concrete due to inertial loading as compared to monotonic reference test ultimate loads.
Table 7.1 Summary of maximum anchor load and crack width at maximum load

<table>
<thead>
<tr>
<th>Anchor ID</th>
<th>N4 Epoxy</th>
<th>N5 Expansion</th>
<th>N7 Drop-In</th>
<th>N9 Undercut</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test Version</td>
<td>V1</td>
<td>V1</td>
<td>V1</td>
<td>V2</td>
</tr>
<tr>
<td>Max. Crack Width [mm]</td>
<td>0.0 mm</td>
<td>0.8 mm</td>
<td>0.0 mm</td>
<td>0.5 mm</td>
</tr>
<tr>
<td>Target Motion Scale [%]</td>
<td>30</td>
<td>30</td>
<td>150</td>
<td>100</td>
</tr>
<tr>
<td>Actual Motion Scale [%]</td>
<td>41</td>
<td>37</td>
<td>152</td>
<td>120</td>
</tr>
<tr>
<td>N_{um,uncr} (Monotonic) [kip]</td>
<td>8.762</td>
<td>8.860</td>
<td>8.920</td>
<td>8.496</td>
</tr>
<tr>
<td>N_{um,cr} (Monotonic) [kip]</td>
<td>n/a</td>
<td>4.353</td>
<td>n/a</td>
<td>6.541</td>
</tr>
<tr>
<td>N_{um,cr} / N_{um,uncr}</td>
<td>0.49</td>
<td>0.77</td>
<td>0.70</td>
<td>0.44</td>
</tr>
<tr>
<td>Failure Type</td>
<td>Breakout</td>
<td>Pullthrough</td>
<td>Pullout/CC</td>
<td>Breakout</td>
</tr>
<tr>
<td>w @ Nmax, NW [mm]</td>
<td>0.00</td>
<td>0.17</td>
<td>0.00</td>
<td>0.09</td>
</tr>
<tr>
<td>w @ Nmax, NE [mm]</td>
<td>0.00</td>
<td>0.29</td>
<td>0.00</td>
<td>0.13</td>
</tr>
<tr>
<td>w @ Nmax, SW [mm]</td>
<td>0.00</td>
<td>0.52</td>
<td>0.00</td>
<td>0.97</td>
</tr>
<tr>
<td>w @ Nmax, SE [mm]</td>
<td>0.00</td>
<td>0.54</td>
<td>0.00</td>
<td>0.44</td>
</tr>
<tr>
<td>N_{max} / N_{um,ref}, NW</td>
<td>unitless</td>
<td>0.95</td>
<td>1.75</td>
<td>1.16</td>
</tr>
<tr>
<td>N_{max} / N_{um,ref}, NE</td>
<td>unitless</td>
<td>0.80</td>
<td>1.26</td>
<td>1.07</td>
</tr>
<tr>
<td>N_{max} / N_{um,ref}, SW</td>
<td>unitless</td>
<td>0.96</td>
<td>0.89</td>
<td>1.00</td>
</tr>
<tr>
<td>N_{max} / N_{um,ref}, SE</td>
<td>unitless</td>
<td>1.10</td>
<td>0.92</td>
<td>0.90</td>
</tr>
</tbody>
</table>

Bold text indicates anchor "failure". Failure is defined as formation of concrete cone or excessive displacement.

Table 7.2 summarizes the ratio of the maximum load achieved in cyclically cracked concrete to corresponding monotonic mean ultimate reference strength ($\Lambda_{\text{exp}} = N_{\text{max}} / N_{\text{um,ref}}$).

Considering all four anchors, $\Lambda_{\text{exp}}$ is on average 1.18 with a coefficient of variation of 0.29.

Considering only the failed anchors, $\Lambda_{\text{exp}}$ is on average 1.0 with a coefficient of variation of 0.32.

This observation suggests that the monotonic reference test may be used as an average predictor of the maximum load achieved under inertial seismic loads in cyclic cracked concrete.
Table 7.2 Ratio of maximum dynamic anchor load in cyclic cracked concrete to monotonic mean reference ultimate load

<table>
<thead>
<tr>
<th>Test</th>
<th>Unfailed Pair</th>
<th>Failed Pair</th>
<th>All four anchors</th>
</tr>
</thead>
<tbody>
<tr>
<td>WALLE.F.N4.CC08.FM02.30P</td>
<td>1.75</td>
<td>0.89</td>
<td>0.92</td>
</tr>
<tr>
<td>V2.WALLE.F.N5.CC05.FM02.100P</td>
<td>1.12</td>
<td>1.06</td>
<td>0.99</td>
</tr>
<tr>
<td>WALLE.F.N5.CC08.FM02.125P</td>
<td>1.17</td>
<td>0.87</td>
<td>0.79</td>
</tr>
<tr>
<td>WALLE.F.N7.CC08.FM02.100P</td>
<td>0.98</td>
<td>0.59</td>
<td>0.54</td>
</tr>
<tr>
<td>WALLE.F.N9.CC08.FM02.60P</td>
<td>1.21</td>
<td>1.75</td>
<td>1.16</td>
</tr>
<tr>
<td>V2.WALLE.F.N9.CC08.FM02.60P</td>
<td>1.51</td>
<td>1.29</td>
<td>1.09</td>
</tr>
</tbody>
</table>

Summary Statistics

<table>
<thead>
<tr>
<th>Statistical Parameter</th>
<th>Unfailed Pair</th>
<th>Failed Pair</th>
<th>All four anchors</th>
</tr>
</thead>
<tbody>
<tr>
<td>mean (μ)</td>
<td>1.18</td>
<td>1.00</td>
<td>1.18</td>
</tr>
<tr>
<td>COV</td>
<td>0.29</td>
<td>0.32</td>
<td>0.29</td>
</tr>
<tr>
<td>max</td>
<td>1.79</td>
<td>1.75</td>
<td>1.79</td>
</tr>
<tr>
<td>min</td>
<td>0.98</td>
<td>0.54</td>
<td>0.54</td>
</tr>
<tr>
<td>σ (σ)</td>
<td>0.25</td>
<td>0.32</td>
<td>0.34</td>
</tr>
<tr>
<td>μ + σ</td>
<td>1.62</td>
<td>1.32</td>
<td>1.52</td>
</tr>
<tr>
<td>μ − σ</td>
<td>1.12</td>
<td>0.67</td>
<td>0.84</td>
</tr>
</tbody>
</table>

Figure 7.19 summarizes the maximum average load on a pair of anchors for each anchor type for cyclic cracked tests and uncracked tests, where applicable for the epoxy and expansion anchors. All anchors except for the drop in anchors performed approximately equally experiencing maximum average loads between 6.5 and 7.7 kips, with average dynamic reduction factors of 0.74-0.75 for the epoxy and expansion anchors respectively. The drop-in anchor sustained a maximum average load of 3.3 kips, 50% of the other anchor types tested. The undercut anchor anchor sustained a maximum average load of 7.7 kips, which is comparable to the epoxy and expansion anchors.
**7.3.2 Displacement Behavior**

The ultimate displacement capacity of an anchor is currently ignored in seismic design of anchorage in the United States (ACI 318, 2008). Based on the results presented below; however, displacement capacity of the anchor is a very important parameter when determining anchor performance for floor anchored components particularly when the anchor is the yielding element. Figure 7.20 summarizes the load-displacement ($N$-$\delta$) relationship for various anchor types during the failure test series. The input amplitude scale at which failure occurred is noted. The expansion and drop-in anchors failed at the highest amplitude floor motion scale, 125% and 100% respectively. Because of their failure mechanisms, these anchors also have the largest displacement capacity, as compared with anchors that fail by concrete breakout (epoxy and undercut), which failed at 30% and 60% amplitude input motion scales respectively.
Figure 7.20 $N$-$\delta$ results for failure test series at increasing amplitude scale conducted in cyclic cracks having a maximum crack width of 0.8mm
Table 7.3 summarizes the anchor displacements at maximum anchor load, $\delta_{N_{\text{max}}}$.

The monotonic ultimate mean displacements are included for reference, $\delta_{u_{\text{m,uncr}}}$ and $\delta_{u_{\text{m,cr}}}$.

The failed anchors are indicated by bold text with shading. The ratio of the maximum displacement achieved in cyclically cracked concrete to corresponding monotonic mean ultimate reference displacement is also listed ($\delta_{N_{\text{max}}} / \delta_{u_{\text{m,ref}}}$). This ratio provides a general indication of the displacements in cyclic cracked concrete due to inertial loading as compared to monotonic reference test ultimate displacements.

Table 7.3 Summary of anchor displacement at maximum anchor load

<table>
<thead>
<tr>
<th>Anchor ID</th>
<th>N4 Epoxy</th>
<th>N5 Expansion</th>
<th>N7 Drop-In</th>
<th>N9 Undercut</th>
</tr>
</thead>
<tbody>
<tr>
<td>Anchor Type</td>
<td>V1</td>
<td>V1</td>
<td>V1</td>
<td>V2</td>
</tr>
<tr>
<td>Test Version</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Max. Crack Width [in]</td>
<td>0.0 mm</td>
<td>0.8 mm</td>
<td>0.0 mm</td>
<td>0.5 mm</td>
</tr>
<tr>
<td>$\delta_{u_{\text{m,uncr}}}$ (Monotonic) [in]</td>
<td>0.025</td>
<td>0.562</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$\delta_{u_{\text{m,cr}}}$ (Monotonic) [in]</td>
<td>0.043</td>
<td>0.562</td>
<td>0.500</td>
<td>0.504</td>
</tr>
<tr>
<td>Failure Type</td>
<td>Breakout</td>
<td>Pullthrough</td>
<td>Pullout/CC</td>
<td>Breakout</td>
</tr>
<tr>
<td>$\delta_{N_{\text{max}}}$, NW [in]</td>
<td>0.006</td>
<td>0.029</td>
<td>0.143</td>
<td>0.197</td>
</tr>
<tr>
<td>$\delta_{N_{\text{max}}}$, NE [in]</td>
<td>0.008</td>
<td>0.035</td>
<td>0.141</td>
<td>0.209</td>
</tr>
<tr>
<td>$\delta_{N_{\text{max}}}$, SW [in]</td>
<td>0.197</td>
<td>0.056</td>
<td>0.075</td>
<td>0.500</td>
</tr>
<tr>
<td>$\delta_{N_{\text{max}}}$, SE [in]</td>
<td>0.013</td>
<td>0.036</td>
<td>0.073</td>
<td>0.401</td>
</tr>
</tbody>
</table>

Bold text indicates anchor "failure". Failure is defined as formation of concrete cone or excessive displacement.

Table 7.4 summarizes the ratio of the anchor displacement at maximum anchor load ($\delta_{N_{\text{max}}}$) achieved in cyclically cracked concrete to corresponding monotonic mean ultimate displacement ($\Lambda_{\delta_{\text{cyclic}}} = \delta_{N_{\text{max}}} / \delta_{u_{\text{m,ref}}}$). Considering all four anchors, $\Lambda_{\delta_{\text{cyclic}}}$ is on average 0.89 with a coefficient of variation of 0.77. Considering only the failed anchors, $\Lambda_{\delta_{\text{cyclic}}}$ is on average
1.2 with a coefficient of variation of 0.70. This observation suggests that the ultimate displacement obtained from the monotonic reference test is unconservative as a predictor of the displacement at maximum load achieved under inertial seismic loads in cyclic cracked concrete; however, the scatter is very large. The coefficient of variation of ultimate displacement from the monotonic reference tests was 0.32.

Table 7.4 Ratio of maximum dynamic anchor displacement at maximum load in cyclic cracked concrete to monotonic mean reference ultimate displacement

<table>
<thead>
<tr>
<th>Test</th>
<th>δ_{N_{max}} / δ_{u_{m,ref}}</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Unfailed Pair</td>
</tr>
<tr>
<td>WALLE.F.N4.CC08.FM02.30P</td>
<td>0.67</td>
</tr>
<tr>
<td>V2.WALLE.F.N5.CC05.FM02.100P</td>
<td>0.39</td>
</tr>
<tr>
<td>WALLE.F.N5.CC08.FM02.125P</td>
<td>0.50</td>
</tr>
<tr>
<td>WALLE.F.N7.CC08.FM02.100P</td>
<td>0.53</td>
</tr>
<tr>
<td>WALLE.F.N9.CC08.FM02.60P</td>
<td>0.39</td>
</tr>
<tr>
<td>V2.WALLE.F.N9.CC08.FM02.60P</td>
<td>0.27</td>
</tr>
</tbody>
</table>

Summary Statistics

<table>
<thead>
<tr>
<th>Statistical Parameter</th>
<th>Unfailed Pair</th>
<th>Failed Pair</th>
<th>All four anchors</th>
</tr>
</thead>
<tbody>
<tr>
<td>mean (μ)</td>
<td>0.58</td>
<td>1.20</td>
<td>0.89</td>
</tr>
<tr>
<td>COV</td>
<td>0.44</td>
<td>0.70</td>
<td>0.77</td>
</tr>
<tr>
<td>max</td>
<td>1.17</td>
<td>2.97</td>
<td>2.97</td>
</tr>
<tr>
<td>min</td>
<td>0.27</td>
<td>0.41</td>
<td>0.27</td>
</tr>
<tr>
<td>sigma (σ)</td>
<td>0.26</td>
<td>0.84</td>
<td>0.68</td>
</tr>
<tr>
<td>μ + σ</td>
<td>0.84</td>
<td>2.04</td>
<td>1.57</td>
</tr>
<tr>
<td>μ − σ</td>
<td>0.32</td>
<td>0.36</td>
<td>0.20</td>
</tr>
</tbody>
</table>

Figure 7.21 summarizes anchor displacement at maximum anchor load for the various anchor types in uncracked and cracked concrete (i.e. graphical representation of data in Table 7.4). The displacement results from dynamic testing follow a similar trend as the monotonic reference testing, namely that anchors which fail by pull-through (expansion) and pullout (drop-in) have the largest displacement 0.2” to 0.6” at maximum anchor load. The anchors that fail by concrete breakout (epoxy and undercut) have a much smaller displacement at maximum load, between 0” to 0.2”. 
Figure 7.21 Anchor displacement, \( \delta \), at maximum anchor load, \( N_{\text{max}} \)

![Anchor Type & Crack Width](image)

Figure 7.22 summarizes anchor displacement at 50 percent of maximum anchor load for the various anchor types in uncracked and cracked concrete. The displacement results at 50 percent of maximum anchor load from dynamic testing follow a similar trend as the monotonic reference testing, namely that anchors which fail by pull-through (expansion) and pullout (drop-in) have the largest displacement 0.1” to 0.4” at maximum anchor load. The anchors that fail by concrete breakout (epoxy and undercut) have a much smaller displacement at maximum load, between 0” to 0.1”. 
7.4 System Performance

System performance of an NCS and for WALLE can be characterized by evaluating the acceleration and displacement response of the component, as well as maximum seismic demand the system resists at anchor failure. It is clear that anchor load-displacement characteristics will affect the nonlinear response of WALLE, however, to what degree is studied in the following sections.

7.4.1 Effect of anchor behavior on system resistance

One measure of seismic resistance is the amplitude scale of floor motion input that caused anchorage failure. A larger amplitude scale at failure indicates that the anchored NCS system is more seismically robust. Figure 7.23 summarizes the actual motion amplitude scale
that caused anchor failure for different anchor types and crack boundary conditions. The anchors that fail by concrete breakout had the lowest input amplitude scale factors. For example the epoxy (un-cracked and cracked) and undercut anchors, which fail by concrete breakout cone, failed at scale factors of 41%, 37%, and 72% respectively. The expansion anchor, which failed by pull-through had the highest seismic resistance with floor motion amplitude scales of 152%, 120% and 120%. This demonstrates the importance of the ultimate displacement capacity of the anchor.

![Figure 7.23 Floor reference motion amplitude re-evaluated scale during final failure test](image)

Figure 7.23 Floor reference motion amplitude re-evaluated scale during final failure test

To further illustrate the importance of anchor displacement capacity on seismic resistance, anchor displacement capacity is plotted versus input motion scale in Figure 7.24. Ultimate displacement capacity $\delta_{um,cr,ref}$ was determined from the monotonic reference tests conducted in cracked concrete having a static crack width equal to 0.8 mm (and 0.5 mm for the expansion anchor). This plot further illustrates that amplitude scale at failure increases as $\delta_{um,cr,ref}$
increases. This further supports the idea that for floor mounted anchored components, displacement capacity of the anchor is desirable to minimize the likelihood of anchorage failure under earthquake loads.

In addition to motion scale, the spectral acceleration at the fundamental period of WALLE during failure tests is also of interest, as it provides a measure of the input motion intensity that physically relates to WALLE response and thus anchor loading. For simplicity the $S_a$ values are calculated at $T_{WALLE} = 0.25$ seconds and assuming $\zeta = 2\%$. The spectra used in these calculations is that associated with the actual achieved motion on the slab at failure. The anchors with the largest displacement capacity (expansion and drop-in) were able to sustain the largest spectral accelerations of approximately 10 g’s (Figure 7.25). The word “sustain” may be slightly misleading in this case because as these anchors displaced, the WALLE period elongates.
from its initial period of 0.25 seconds and even though the anchor load capacity has decreased, the input motion intensity can still be sustained by the WALLE acting in an “unanchored-rocking” mechanism without the WALLE actually tipping over. In other words, since WALLE is mounted vertically, gravity and its large weight provides some natural recentering capacity. As shown in previously by the unanchored rocking tests, WALLE period can elongate to a period of 1 seconds or greater, where the spectral demand for the FM02 motion maybe many times less that the spectral demand at 0.25 seconds.

**Figure 7.25** Floor motion 2% damped spectral acceleration, g, at the initial period of WALLE during the final failure test

Figure 7.26 compares the Peak Input Acceleration (PIA) with the Peak WALLE Acceleration (PWA) at the center of mass of WALLE. In this case PIA is taken as the peak acceleration measured at the slab, while PWA is taken as the interpolated peak value at the center of mass. The data shown includes only the failure motions. Two very clear trends can be
observed, the first being that the PWA was much larger for the uncracked concrete case. For example PWA (epoxy, uncracked) was approximately 1.75 g’s and the cracked case for the same anchor located in cyclic cracked concrete PWA (epoxy, \(w=0.8\) mm) was slightly above 1g (approximately 40% reduction in acceleration for the same anchor located in cyclic cracks. A reduction in PWA also occurs for the expansion anchors, where the PWA in uncracked concrete was approximately 3g, while the PWA for cyclic cracked concrete was approximately 2g (33% reduction). The second trend is that the Peak Input Acceleration at anchor failure was larger for the expansion and drop-in anchors, both of which have failure mechanisms that allow for more displacement before a loss of load capacity than the other anchors, which failed by concrete breakout. Figure 7.27 summarizes the results from all failure test series. It is again noted that the uncracked concrete case causes a larger acceleration amplification for the same anchor type.

Figure 7.26 Peak WALLE acceleration (PWA) versus peak input acceleration (PIA), final failure test
It is interesting, however, important to note that amplification of the motion occurs in 89% of the cases (most of the time). This can be investigated by plotting the component acceleration amplification, as defined by Peak WALLE Acceleration over Peak Input Acceleration (PWA/PIA) (Figure 7.28). It is important to note that this ratio is uncorrelated, i.e. the maximum WALLE acceleration need not occur at the same instant in time as the maximum input acceleration. The uncracked tests for epoxy and expansion anchor display the largest component amplifications, with values of 3.0 and 2.5, respectively. The WALLE/anchor systems tested in cracked concrete all had component amplifications less than 2.0. These results suggest that the system softening and associated period elongation that takes place due to anchors being located in cracked concrete of anchor displacement is beneficial in reducing the acceleration amplifications that are transferred to the component. Figure 7.29 summarizes the component amplification for all failure test series. The component amplification ranged from 0.5 to 2.0 for
the cyclic cracked test and from approximately 2.0 to 3.5 for the uncracked tests. The data in Figure 7.29 also show that in most cases (77%) of the pre-failure and failure simulations, result in acceleration amplification of the model NCS.

Figure 7.28 Acceleration amplification factor, final failure tests
7.5 Summary Remarks

Based on experimental observations and measurements related to the response of the anchored WALLE system, the following conclusions can be made:

- The component amplification calculated as the ratio of the measured uncorrelated peak WALLE acceleration at the center of mass and the peak input acceleration, ranged from 1.0 to 2.0 for the cyclic cracked failure tests and from approximately 2.0 to 3.5 for the uncracked failure tests.

- Anchor forces derived from an equilibrium analysis using WALLE experimental acceleration and displacement history corresponded well with the experimentally measured anchor load histories. These analyses associated with assessing the robustness of the anchor load washer measurements.

Figure 7.29 Acceleration amplification factor versus anchor type, all failure tests series of increasing motion amplitude to failure
The average predicted maximum anchor load using a response spectrum analysis approach observes significant scatter, with estimations ranging from approximately one-half to two times that measured.

Anchor forces are not evenly distributed between a pair of anchors even for uniaxial seismic loading of a relatively simple model SDOF NCS system. The load distribution between anchors in these tests observed to be highly variable at low to moderate loading amplitudes. The load distribution between two anchors on a side tends to even out at large loads close to failure. The coefficient of variation of maximum anchor force was 50% considering all failure tests series of increasing motion amplitude up to and including the final failure tests. The coefficient of variation was 35% considering only the final failure tests.

The achieved anchor load amplitude that caused anchor failure for the tests conducted in cracked cyclic concrete was generally approximately equal to $N_{um,cr}$ obtained from monotonic reference tests in statically open cracks. Therefore in this aspect, the experimental tests support the current code philosophy of using the reference anchor monotonic strength in cracked concrete with statically open cracks (non-cycled cracks). The exception to this was the drop-in anchors, which failed in the dynamic tests at approximately 30 to 60% $N_{um,cr}$ further supporting the philosophy that anchors that are not seismically qualified for use in cracked concrete should not be used in seismic applications.

Increasing input motion scale at failure, which is an indicator of seismic resistance, was directly correlated with increasing anchor ultimate displacement capacity $\delta_u$ for the floor mounted anchored WALLE. This agrees with the current code design philosophy that, for high seismic applications, for NCSs that remain elastic and do not in themselves have energy absorption capacity, either the anchorage should be designed to have adequate
displacement capacity (ductility) or the attachment should be designed as the yielding element at a load less than the anchor capacity. Alternately, the anchors should be designed for overstrength forces of the attachment or the NCS.

- The maximum achieved anchor load amplitude was generally between $N_{um,cr}$ and $N_{um,uncr}$ for the tests conducted in cracked cyclic concrete.

- The WALLE system which observed anchorage that failed by pull-through (expansion) provides significant lateral displacement capacity with an ultimate component drift ratio of 5.5%. The WALLE system with concrete breakout (epoxy) anchors has a reduced lateral displacement capacity with an ultimate component drift ratio of 1.5%.

- The ultimate load $N_{um,cr}$ from monotonic reference testing in cracked concrete having a static crack width $w_{max}$ may be used as an average predictor of the maximum load achieved at anchor failure under inertial seismic loads in cyclic cracked concrete.

- The ultimate displacement obtained from the monotonic reference testing $\delta_{um,cr}$ is unconservative when compared to the displacement at maximum load achieved under inertial seismic loads in cyclic cracked concrete. These tests suggest $\delta_{um,cr}$ is approximately 1.2 times the displacement at maximum anchor load in cyclically cracked concrete; however the scatter is very large, $CV=0.7$ therefore this finding is still within the typical range of scatter on displacement.

- A large ultimate displacement capacity of the anchors is desirable for increasing the seismic resistance of components where the component remains elastic; however, the increased component displacement may be an unacceptable tradeoff for displacement sensitive NCSs.

- It was experimentally observed that anchor preload was typically lost within the first 3-5 strong cycles of anchor loading. This suggests that if anchor preload is important to
component forces then the status of anchor preload should be field verified after even small to moderate seismic events.
Chapter 8 Experimental Results: Correlation Test Series

8.1 Introduction

The tension load carrying capacity of an anchor reduces due to the presence of a crack in the surrounding base concrete. It is important therefore to understand, under real seismic loading conditions, the amplitude of the load in an anchor at a particular crack width. To investigate the relationship between tension load amplitude ($N$) and crack width ($w$) coincidence a series of variable phase and in-phase correlation testing were performed. In this test series, anchors are purposely loaded to less than half of their mean ultimate load capacity in cracked concrete, while cracks are cycled with a maximum crack width of 0.8 mm [i.e. $50\% N_{um,cr}$ ($w_{max}$=0.8mm)]. The test setup is similar to that described in the failure test series and is described in Chapter 4.

Important issues to consider when investigating the correlation between load amplitude and crack width include the following:

- **Input ground motion characteristics**: including amplitude, frequency content, number of acceleration cycles and strong shaking duration;

- **Structure**: Dynamic characteristics of the primary structure filter the input ground motion, therefore, its geometry, number of stories, lateral load resisting system, and non-linear load-deformation characteristics are important. In addition, the primary structure manifests cyclic curvature, and hence flexural crack histories, which are input to the component (NCS) anchorage;

- **Component**: Dynamic characteristics of the Nonstructural Component or System (NCS) including, natural period, damping, inelastic properties, and location of the NCS within the primary structure, will each affect the demands on the anchors;
- **Anchor**: Properties of the anchor itself that fastens the NCS to the primary structure including stiffness, strength, deformation and ductility capacity.

As a result of the many factors above and their inherent variability, the problem of anchor loading and crack width coincidence is a very complex. Yet, for it to be considered in design, actual field conditions from one site-building-NCS combination to the next must be generalized. In this work, we design a test matrix with variables selected to study most of the influencing factors described above (Table 8.1). An effort was made to select variations that may be expected in practice. It should be noted that for practical reasons, nonlinearity of the NCS was not considered in the program.
Table 8.1 Input parameters considered in the correlation test series

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Variations</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Input ground motion</strong></td>
<td>A suite of 8 ground motions with varying magnitude distance pairs and faulting styles were input to nonlinear building simulations (described below)</td>
</tr>
<tr>
<td><strong>Name</strong></td>
<td><strong>Location</strong></td>
</tr>
<tr>
<td>Baja</td>
<td>Mexico</td>
</tr>
<tr>
<td>Friuli</td>
<td>Italy</td>
</tr>
<tr>
<td>Irpina</td>
<td>Italy</td>
</tr>
<tr>
<td>Kobe</td>
<td>Japan</td>
</tr>
<tr>
<td>Northridge1</td>
<td>California</td>
</tr>
<tr>
<td>Northridge2</td>
<td>California</td>
</tr>
<tr>
<td>SanFernando</td>
<td>California</td>
</tr>
<tr>
<td>Superstition</td>
<td>California</td>
</tr>
<tr>
<td><strong>Ground motion amplitude</strong></td>
<td>The input ground motions were linearly scaled to achieve target anchor tension forces between 10 and 50% of the ultimate tensile capacity in cracked concrete</td>
</tr>
<tr>
<td></td>
<td>• Scale factors ranging from 6% to 120% of the maximum credible earthquake (MCE)</td>
</tr>
<tr>
<td><strong>Building height</strong></td>
<td>The dynamic and nonlinear characteristics of the buildings were varied by considering the following reinforced concrete special moment resisting frames with 2, 4, 8, 12, and 20 stories.</td>
</tr>
<tr>
<td><strong>Location of NCS within building</strong></td>
<td>• Floor = 1, 2, 3, 4, 5, 6, 9, 10, 18</td>
</tr>
<tr>
<td></td>
<td>• z/h = 0.13, 0.17, 0.20, 0.25, 0.38, 0.42, 0.45, 0.50, 0.63, 0.83, 0.90</td>
</tr>
<tr>
<td></td>
<td>z = elevation of NCS, h = elevation of the roof</td>
</tr>
<tr>
<td></td>
<td>(measured from the ground to the point of attachment of the NCS)</td>
</tr>
<tr>
<td><strong>NCS period</strong></td>
<td>• Rigid NCS, $T_{\text{NCS}} = 0.25 \text{ sec}$</td>
</tr>
<tr>
<td></td>
<td>• Flexible NCS, $T_{\text{NCS}} = 0.10 \text{ sec}$</td>
</tr>
<tr>
<td><strong>Anchor type</strong></td>
<td>Undercut anchors (N9) were used for the variable phase and in-phase correlation tests.</td>
</tr>
<tr>
<td><strong>Effective duration of strong floor acceleration</strong></td>
<td>Durations ranging from $t_d = 6$ to 28 sec. as measured by time from 5 to 95% arias intensity</td>
</tr>
<tr>
<td><strong>Predominant period of crack cycling</strong></td>
<td>$T_{\text{p,cr}} = 0.24, 0.47, 0.89, 1.33, 2.07 \text{ sec}$</td>
</tr>
</tbody>
</table>

8.2 Application to Qualification Testing

Qualification tests are required to determine the suitability of an anchor for use in seismic applications in cracked concrete. The three main seismic qualification test types associated with anchors in tension in cracked concrete are: i) monotonic tension loading with constant crack
width, ii) tension load cycling with constant crack width and iii) crack cycling with constant (or varying) applied tension load history [ACI 355.2, 2007; Mahrenholtz, 2009]. The selection of a suitable load history for use while performing crack cycling tests (case iii) remains an open question. One option is to use a constant load that is a fraction of the maximum load that occurs during typical seismic applications. To determine the fraction of load that should be applied during qualification tests, mean load level statistics from the correlation test series are calculated over a range of crack widths. The actual constant load to be applied during testing can be set at a percentage above the mean load depending on what level of confidence of non-exceedance is desired. The mean load is described by a percentage of the maximum achieved load or ultimate load of the anchor. The load percentage is herein called \( \alpha_{cc} \) for alpha “crack cycling” which can be defined by the following equation:

\[
\alpha_{cc} = \frac{N_m}{N_{um,cr}} = \frac{\text{Mean applied load}}{\text{Mean failure load in cracked concrete}}
\]

In this work, the parameter \( \alpha_{cc} \) is determined for individual crack width bins. The bin sizes and binning procedure is described in detail in a later section.

### 8.3 Test Groupings

A summary of the test matrix was provided in Chapter 4. To observe trends in the data, tests were grouped for further study (Table 1.2 and Table 8.3). Data from the variable phase correlation tests, in-phase correlation tests and the failure tests are incorporated into this study. The following are the parameters used in determining the test groups:

- Anchor type
• Failure test versus correlation test
• Flexible versus stiff WALLE
• Building height (indirectly accounts for the phasing between $T_{WALLE}$ and $T_I$ of the building and predominant period of loading $T_N$ and period of cracking $T_{p,cr}$)
• “In-phase” versus “variable phase” tests
266
Table 8.2 Individual test names and test groups

1
1
1
1
1
1
2
2
1
3
5
5
2
5
6
10
4
4
9
18
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5
6
10
4
4
9
18
18
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1
n/a
n/a
n/a
5
5
5

SUM.CORR.R.N9.CC08.Under.2ST

SUM.CORR.R.N9.CC08.Undercut

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2st
4st
4st
4st
4st
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8st
8st
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12st
12st
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D=0.25", 1Hz
D=0.25", 1Hz
D=0.1", FM14
D=0.25", FM14
D=0.75", FM14

SUM.CORR.R.N9.CC08.Undercut.4ST

SUM.CORR.R.N9.CC08.Undercut.8ST

SUM.CORR.R.N9.CC08.Undercut.12ST

SUM.CORR.R.N9.CC08.Undercut.20ST

SUM.CORR.F.N9.CC08.Undercut.2ST

SUM.CORR.F.N9.CC08.Undercut

irpina
kobe00
north
sanf
super
nga
friuli
north
irpina
friuli
baja
kobe00
super
super
baja
kobe00
nga
kobe00
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super.12st
super.12st

SUM.ALL.CORRELATION

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Test Groups

SUM.CORR.F.N9.CC08.Undercut.4ST

SUM.CORR.F.N9.CC08.Undercut.8ST

SUM.CORR.F.N9.CC08.Undercut.12ST

SUM.CORR.F.N9.CC08.Undercut.20ST

SUM.N4.CC08.Epoxy

SUM.N5.CC05.V1.Expansion
SUM.N5.CC05.ALL.Expansion
SUM.N5.CC05.V2.Expansion

SUM.ALL.FAILURE

Floor

V2.WALLE.R.N9.CC08.FM01
V2.WALLE.R.N9.CC08.FM02
V2.WALLE.R.N9.CC08.FM03
V2.WALLE.R.N9.CC08.FM04a
V2.WALLE.R.N9.CC08.FM05a
V2.WALLE.R.N9.CC08.FM06a
V2.WALLE.R.N9.CC08.FM07
V2.WALLE.R.N9.CC08.FM08
V2.WALLE.R.N9.CC08.FM09
V2.WALLE.R.N9.CC08.FM10
V2.WALLE.R.N9.CC08.FM11a
V2.WALLE.R.N9.CC08.FM12
V2.WALLE.R.N9.CC08.FM13
V2.WALLE.R.N9.CC08.FM14
V2.WALLE.R.N9.CC08.FM15
V2.WALLE.R.N9.CC08.FM16
V2.WALLE.R.N9.CC08.FM17
V2.WALLE.R.N9.CC08.FM18
V2.WALLE.R.N9.CC08.FM19
V2.WALLE.R.N9.CC08.FM20
V2.WALLE.F.N9.CC08.FM01
V2.WALLE.F.N9.CC08.FM02
V2.WALLE.F.N9.CC08.FM03
V2.WALLE.F.N9.CC08.FM04
V2.WALLE.F.N9.CC08.FM05
V2.WALLE.F.N9.CC08.FM06
V2.WALLE.F.N9.CC08.FM07
V2.WALLE.F.N9.CC08.FM08
V2.WALLE.F.N9.CC08.FM09
V2.WALLE.F.N9.CC08.FM10
V2.WALLE.F.N9.CC08.FM11
V2.WALLE.F.N9.CC08.FM12
V2.WALLE.F.N9.CC08.FM13
V2.WALLE.F.N9.CC08.FM14
V2.WALLE.F.N9.CC08.FM15
V2.WALLE.F.N9.CC08.FM16
V2.WALLE.F.N9.CC08.FM17
V2.WALLE.F.N9.CC08.FM18
V2.WALLE.F.N9.CC08.FM19
V2.WALLE.F.N9.CC08.FM20
V2.WALLE.F.N9.CC08.FM20.36P
WALLE.F.N4.CC08.FM02.15P
WALLE.F.N4.CC08.FM02.20P
WALLE.F.N4.CC08.FM02.30P
WALLE.F.N5.CC05.FM02.10P
WALLE.F.N5.CC05.FM02.30P
WALLE.F.N5.CC05.FM02.100Pa
V2.WALLE.F.N5.CC05.FM02.10P
V2.WALLE.F.N5.CC05.FM02.30P
V2.WALLE.F.N5.CC05.FM02.100P
WALLE.F.N5.CC08.FM02.10P
WALLE.F.N5.CC08.FM02.30P
WALLE.F.N5.CC08.FM02.100P
WALLE.F.N5.CC08.FM02.125P
WALLE.F.N7.CC08.FM02.5P
WALLE.F.N7.CC08.FM02.15P
WALLE.F.N7.CC08.FM02.20P
WALLE.F.N7.CC08.FM02.25P
WALLE.F.N7.CC08.FM02.35P
WALLE.F.N7.CC08.FM02.60P
WALLE.F.N7.CC08.FM02.100P
WALLE.F.N9.CC08.FM02.15P
WALLE.F.N9.CC08.FM02.20P
WALLE.F.N9.CC08.FM02.40Pa
WALLE.F.N9.CC08.FM02.60P
V2.WALLE.F.N9.CC08.FM02.20P
V2.WALLE.F.N9.CC08.FM02.40P
V2.WALLE.F.N9.CC08.FM02.60P
STRUCT.N9.CC08.CP2.D1a
STRUCT.N9.CC08.CP2.D1b
STRUCT.N9.CC08.CP2.D1c
STRUCT.N9.CC08.CR14.D14a
STRUCT.N9.CC08.CR14.D14c

Building

SUM.ALL.TESTS

64
78
81
69
68
73
79
82
62
67
60
66
72
80
65
77
63
70
71
61
92
101
105
98
87
94
91
104
88
99
93
103
89
96
90
97
100
102
95
107
106
130
131
132
123
124
126
144
145
146
54
55
56
57
135
136
137
138
139
140
141
48
49
50
51
118
119
120
174
175
176
177
178
179

Motion
Name

(mm)

Test Name

wmax

No.

Input Motion Parameters

SUM.N5.CC08.Expansion

SUM.N7.CC08.RDIDrop-In

SUM.N9.CC08.V1.Undercut
SUM.N9.CC08.ALL.Undercut
SUM.N9.CC08.V2.Undercut

SUM.STRUCT.1HZ

SUM.ALL.STRUCT
SUM.STRUCT.FM14


Table 8.3 Test groups

<table>
<thead>
<tr>
<th>Group</th>
<th>Group Name</th>
<th>Description</th>
<th>No. Tests</th>
</tr>
</thead>
<tbody>
<tr>
<td>G1</td>
<td>SUM.ALL.TESTS</td>
<td>Sum of G1 to G27</td>
<td>68</td>
</tr>
<tr>
<td>G2</td>
<td>SUM.ALL.CORRELATION</td>
<td>Sum of G7 to G16</td>
<td>41</td>
</tr>
<tr>
<td>G3</td>
<td>SUM.ALL.FAILURE</td>
<td>Sum of G17 to G25</td>
<td>27</td>
</tr>
<tr>
<td>G4</td>
<td>SUM.ALL.STRUCT</td>
<td>Sum of G26 to G27</td>
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<tr>
<td>G5</td>
<td>SUM.CORR.R.N9.CC08.Undercut</td>
<td>Sum of G7 to G11</td>
<td>20</td>
</tr>
<tr>
<td>G6</td>
<td>SUM.CORR.F.N9.CC08.Undercut</td>
<td>Sum of G12 to G16</td>
<td>21</td>
</tr>
<tr>
<td>G7</td>
<td>SUM.CORR.R.N9.CC08.2ST</td>
<td>Variable phase correlation tests</td>
<td>4</td>
</tr>
<tr>
<td>G8</td>
<td>SUM.CORR.R.N9.CC08.4ST</td>
<td>By building height (Stiff WALLE)</td>
<td>4</td>
</tr>
<tr>
<td>G9</td>
<td>SUM.CORR.R.N9.CC08.8ST</td>
<td>4</td>
<td></td>
</tr>
<tr>
<td>G10</td>
<td>SUM.CORR.R.N9.CC08.12ST</td>
<td>4</td>
<td></td>
</tr>
<tr>
<td>G11</td>
<td>SUM.CORR.R.N9.CC08.20ST</td>
<td>4</td>
<td></td>
</tr>
<tr>
<td>G12</td>
<td>SUM.CORR.F.N9.CC08.2ST</td>
<td>Variable phase correlation tests</td>
<td>4</td>
</tr>
<tr>
<td>G13</td>
<td>SUM.CORR.F.N9.CC08.4ST</td>
<td>By building height (Flexible WALLE)</td>
<td>4</td>
</tr>
<tr>
<td>G14</td>
<td>SUM.CORR.F.N9.CC08.8ST</td>
<td>4</td>
<td></td>
</tr>
<tr>
<td>G15</td>
<td>SUM.CORR.F.N9.CC08.12ST</td>
<td>4</td>
<td></td>
</tr>
<tr>
<td>G16</td>
<td>SUM.CORR.F.N9.CC08.20ST</td>
<td>4</td>
<td></td>
</tr>
<tr>
<td>G17</td>
<td>SUM.N4.CC08.Epoxy</td>
<td>Failure test series by anchor type</td>
<td>3</td>
</tr>
<tr>
<td>G18</td>
<td>SUM.N5.CC05.ALL.Expansion</td>
<td>Sum of G19 to G20</td>
<td>10</td>
</tr>
<tr>
<td>G19</td>
<td>SUM.N5.CC05.V1.Expansion</td>
<td>3</td>
<td></td>
</tr>
<tr>
<td>G20</td>
<td>SUM.N5.CC05.V2.Expansion</td>
<td>3</td>
<td></td>
</tr>
<tr>
<td>G21</td>
<td>SUM.N5.CC08.Expansion</td>
<td>4</td>
<td></td>
</tr>
<tr>
<td>G22</td>
<td>SUM.N7.CC08.Drop-In</td>
<td>7</td>
<td></td>
</tr>
<tr>
<td>G23</td>
<td>SUM.N9.CC08.ALL.Undercut</td>
<td>Sum of G24 to G25</td>
<td>7</td>
</tr>
<tr>
<td>G24</td>
<td>SUM.N9.CC08.V1.Undercut</td>
<td>4</td>
<td></td>
</tr>
<tr>
<td>G26</td>
<td>SUM.STRUCT.1HZ.Undercut</td>
<td>In-phase correlation tests (load protocol)</td>
<td>3</td>
</tr>
<tr>
<td>G27</td>
<td>SUM.STRUCT.FM14.Undercut</td>
<td>In-phase correlation tests (FM14)</td>
<td>3</td>
</tr>
</tbody>
</table>

8.4 Load and Crack Bin Sizes

It is useful to organize the normalized crack width ($w^*$) and normalized load ($N^*$) into manageable bin sizes for subsequent statistical analysis. Bin sizes were selected as summarized in Table 8.4. Table 8.4 describes the bin limits for load and crack width used in the load versus crack width analysis. Bin zero for the normalized load limit has a lower bound of zero because compression load is not physically possible for the load washers in an in-situ condition; however, the zero bin for the normalized crack width is extended to -0.05 to maintain a centered binning approach and because negative crack width is physically possible as it relates to a closing of the
hairline crack ($w_{\text{hairline}}$). Data in bin 1a is extremely small and is not included in the statistical analysis.

Table 8.4 Bin sizes for normalized parameters

<table>
<thead>
<tr>
<th>Bin No.</th>
<th>Bin Center</th>
<th>Load Limits ($N^*$)</th>
<th>Crack Limits ($w^*$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0.025 / 0.0</td>
<td>0.00 &lt; $N^*$ ≤ 0.05</td>
<td>-0.05 &lt; $w^*$ ≤ 0.05</td>
</tr>
<tr>
<td>1</td>
<td>0.1</td>
<td>0.05 &lt; $N^*$ ≤ 0.15</td>
<td>0.05 &lt; $w^*$ ≤ 0.15</td>
</tr>
<tr>
<td>2</td>
<td>0.2</td>
<td>0.15 &lt; $N^*$ ≤ 0.25</td>
<td>0.15 &lt; $w^*$ ≤ 0.25</td>
</tr>
<tr>
<td>3</td>
<td>0.3</td>
<td>0.25 &lt; $N^*$ ≤ 0.35</td>
<td>0.25 &lt; $w^*$ ≤ 0.35</td>
</tr>
<tr>
<td>4</td>
<td>0.4</td>
<td>0.35 &lt; $N^*$ ≤ 0.45</td>
<td>0.35 &lt; $w^*$ ≤ 0.45</td>
</tr>
<tr>
<td>5</td>
<td>0.5</td>
<td>0.45 &lt; $N^*$ ≤ 0.55</td>
<td>0.45 &lt; $w^*$ ≤ 0.55</td>
</tr>
<tr>
<td>6</td>
<td>0.6</td>
<td>0.55 &lt; $N^*$ ≤ 0.65</td>
<td>0.55 &lt; $w^*$ ≤ 0.65</td>
</tr>
<tr>
<td>7</td>
<td>0.7</td>
<td>0.65 &lt; $N^*$ ≤ 0.75</td>
<td>0.65 &lt; $w^*$ ≤ 0.75</td>
</tr>
<tr>
<td>8</td>
<td>0.8</td>
<td>0.75 &lt; $N^*$ ≤ 0.85</td>
<td>0.75 &lt; $w^*$ ≤ 0.85</td>
</tr>
<tr>
<td>9</td>
<td>0.9</td>
<td>0.85 &lt; $N^*$ ≤ 0.95</td>
<td>0.85 &lt; $w^*$ ≤ 0.95</td>
</tr>
<tr>
<td>10</td>
<td>0.975</td>
<td>0.95 &lt; $N^*$ ≤ 1.00</td>
<td>0.95 &lt; $w^*$ ≤ 1.00</td>
</tr>
</tbody>
</table>

8.5 Procedure for Normalization and Data Processing

Matlab® software was used to read in data from the various test groups and process the load and crack width data accordingly. The following steps were taken in the data processing to normalize quantities so that tests run at different scale factors and different maximum crack widths could be compared with each other:

1) Remove negative loads $N < 0$

2) Normalize the crack width at each anchor “j” at each instant in time “i” $w_{i,j}$ by $\max(w_{i,j})$ to obtain $w^*$ using the $\max(w_{i,j})$ from each anchor NW,NE,SW,SE for (i.e. the maximum $w^*$ for each anchor is 1.0)

3) Delete all $N$-$w$ points with excessively low $N$ values (5% $N_{\text{num,cr}}$ was selected for this example).

4) Normalize $N_i$ by $N_{\text{num,cr}}$ to obtain $N_i/N_{\text{num,cr}}$ also referred to as $\%N_{\text{num,cr}}$.
3) Find the load maximum per each load cycle, $N_{\text{max},k}$ and their corresponding $w_k^*$

$$(N_{\text{max},k} / N_{\text{um,cr}} \text{ vs. } w_k^*)$$

5) Plot all data ($N_i / N_{\text{um,cr}}$ vs. $w_i^*$) and max. loads per cycle ($N_{\text{max},k} / N_{\text{um,cr}}$ vs. $w_k^*$), including averages and +/- one standard deviation error bars

6) Normalize anchor load history $N_i$ for each anchor location $j$, e.g. $N_{ij} / N_{j,\text{max}}$ (to give $N^*$) using the maximum load $N_{j,\text{max}}$ from each anchor (NW,NE,SW,SE) for normalization of that anchor (i.e. $N_{\text{max}}^*=1.0$ for each anchor)

7) Plot all data ($N_i^* \text{ vs. } w_i^*$) and maximum loads per cycle ($N_{\text{max},k}^* \text{ vs. } w_k^*$) averages and one standard deviation error bars

Steps to develop groupings:

8) Combine data from step 7 from each individual test into “like” test groups shown in Table 8.3 (e.g. SUM_ALL_CORRELATION and SUM_ALL_FAILURE)

9) Recalculate average load $N^*$ and one standard deviation error bars using all test data in the test group

10) Plot results from step 9 on “overlay” plots to compare results from different test groups.

### 8.5.1 Example of Normalization Procedure

The following figures explain step-by-step the normalization procedure described above. For demonstration, test number 118, V2.WALLE.F.N9.CC08.FM02.20P from Table 8.2 is used throughout the example. This test is a version 2 (V2) WALLE test in the flexible (F) configuration using an undercut anchor (N9) tested in a maximum crack width of 0.8 mm (CC08) using floor motion two (FM02) scaled to 20% (20P).
8.5.2 Load and crack histories

Figure 8.1 shows the raw anchor load $N$ versus time $t$ measured during the test for each anchor (NW, NE, SW, SE). The load history between the North and South pair of anchors is out of phases as WALLE rocks back and forth from North to South. Figure 8.2 shows an example of the raw crack width $w$ versus time $t$ measured during the test for each anchor (NW, NE, SW, SE). It can be seen that all cracks are cycling in phase and to approximately the same peak amplitude.

![Graphs showing anchor load history](image)

**Figure 8.1 Example anchor tension load history ($N$) for test V2.WALLE.F.N9.CC08.FM02.20P**
8.5.3 Load cycle peaks ($N_{max,k}$) and crack cycle peaks ($w_{max,k}$)

Figure 8.3 demonstrates the difference between all discretely sampled data points, $N_i$, at 200 samples per second are fully utilized and the maximum load per load cycle $N_{max,k}$. The local maximums per load cycle $N_{max,k}$ were calculated by first finding all peaks and valleys over a +/- 2 point window of the point at time “i” under consideration (Standard Matlab® “findpeaks.m” function). The peaks were considered as $N_{max,k}$ as long as the following local maximum criteria was also met. Peak load cycle k ($N_k$) was also checked to determine if the local peak to valley amplitude ($\Delta N_k$) was greater than 10% of the minimum of either the peak amplitude under consideration $N_k$ or the next peak amplitude $N_{k+1}$. If the local peak to valley amplitude $\Delta N_k$ was less than 10% minimum($N_k, N_{k+1}$) then the local maximum was discarded from $N_{max,k}$. For example the local maximum that occurs at 5.94 seconds $\Delta N_k$ has a local peak to valley amplitude
less than 10% of the adjacent peaks \( N_k \) and \( N_{k+1} \) therefore it is not considered as a valid \( N_{\text{max},k} \) peak.

![Figure 8.3 Maximum load per cycle, \( N_{\text{max,k}} \) versus all load points \( N_i \)](image)

Once the maximum loads per cycle \( N_{\text{max,k}} \) are found (Figure 8.4a), the corresponding crack widths \( w_k \) are found, (Figure 8.4b). Similarly, the maximum crack width per crack cycle \( w_{\text{max,k}} \) are found (Figure 8.4d), therefore the corresponding loads \( N_k \) are found (Figure 8.4c). Figure 8.5 combines anchor load and crack width on the same plot for the example test. The maximum anchor loads and corresponding crack widths are identified. It is noted that the crack width at maximum anchor loads was small, or the crack was closed completely.
Figure 8.4 (a) Maximum load per load cycle, $N_{max,k}$ and (b) corresponding crack widths, $w_k$
(c) maximum crack width per crack cycle and (d) $w_{max,k}$ and corresponding loads, $N_k$
Figure 8.5 Example anchor load $N$ and crack width history $w$ showing $N_{\text{max}}$ and the corresponding crack width $w@N_{\text{max}}$ for test V2.WALLE.F.N9.CC08.FM02.20P

Figure 8.6 shows the raw anchor load $N$ versus time $t$ measured during test V2.WALLE.F.N9.CC08.FM02.20P for each anchor (NW, NE, SW, SE). Peak values per cycle $k$ are identified with black dots for each of $N_{\text{max},k}$ and $w_{\text{max},k}$. Figure 8.7 shows the raw crack width $w$ versus time $t$ measured during the test for each anchor (NW, NE, SW, SE). Figure 8.8 repeats a portion of the test windowed in time from 7-9 seconds in greater detail. It is noted that there are several small load peaks for the Southwest anchor; however, when zoomed in closely, these are indeed peaks with a small peak to valley amplitude. Figure 8.9 shows the raw crack width $w$ versus time $t$ measured during the test for each anchor (NW, NE, SW, SE) windowed in on the history from 7-9 seconds. Peak values per cycle $k$ are identified with black dots for each of $N_{\text{max},k}$ and $w_{\text{max},k}$. These plots articulate that because data are selected on a per peak basis, the unload and reload characteristics of the anchor are not retained in the statistics. It is recognized however, that peak data may be sufficient to characterize the most important loading and boundary conditions to the anchor, in recognition that these peak values will have the most detrimental effect on the anchors performance thereby rendering conservative load and crack width statistics.
Figure 8.6 Example anchor tension load history showing $N_{\text{max,k}}^*$ (black dots) for test V2.WALLE.F.N9.CC08.FM02.20P

Figure 8.7 Example crack width history showing $w_{\text{max,k}}^*$ (black dots) for test V2.WALLE.F.N9.CC08.FM02.20P
Figure 8.8 Example anchor tension load history windowed showing $N_{\text{max},k}^*$ (black dots) for test V2.WALLE.F.N9.CC08.FM02.20P

Figure 8.9 Example crack width history windowed showing $w_{\text{max},k}^*$ (black dots) for test V2.WALLE.F.N9.CC08.FM02.20P
8.5.4 Load-crack width relationship

It is instructive to characterize the anchor tension $N$ to crack width $w$ relationship. In this case, Figure 8.10 presents each $N_{i,j} - w_{i,j}$ pair for test V2.WALLE.F.N9.CC08.FM02.20P. The maximum load, maximum crack width and distribution of $N$-$w$ varies for each anchor with a general trend of decreasing $N$ with increasing $w$. The North-West (NW) anchor experienced the largest force, while the South-West (SW) anchor experienced the largest crack width. Figure 8.11 presents data for all four anchors (NW, NE, SW, and SE) overlaid on one plot for this test. By combining data from all four anchors together, the data set becomes fuller, decreasing the aleatory variability. It is also noted that the envelope of the data renders a conservative estimate of the $N$ associated with a given $w$.

![Figure 8.10 N-w pair distribution, individual anchors for test V2.WALLE.F.N9.CC08.FM02.20P, envelope of data shown](image)
8.5.5 Normalization of load and crack width

In order to compare $N$-$w$ results from different tests where $N_{\text{max}}$ and $w_{\text{max}}$ vary due to motion scale or maximum desired crack width, the load and crack width for each anchor are normalized by their maximum values for each anchor, NW, NE, SW, and SE, respectively. As noted previously, normalized anchor axial load and normalized crack width are noted with an asterisk $N^*$ and $w^*$ respectively.

Figure 8.12 shows an example of a load history normalized such that the maximum is 1.0 for each anchor. In other words, each anchor load history is locally normalized such that the maximum value is 1.0. Figure 8.13 shows an example of a crack width history normalized such that the maximum is 1.0 for each anchor. In the following discussions, $N^*$ is used to generally
represent normalized load which may be $N_i^*$ or $N_{\text{max},k}^*$ therefore $N^*$ is used interchangeably in the general sense for normalized load.

![Figure 8.12 Example normalized tension load history ($N^*$) for test V2.WALLE.F.N9.CC08.FM02.20P](image_url)
Figure 8.13 Example of normalized crack width history (w*) for test V2.WALLE.F.N9.CC08.FM02.20P

The normalized N*-w* quantities are plotted against each other to observe trends in the data. Normalized anchor axial load (N*) versus normalized crack width (w*) data are shown for each anchor in Figure 8.14. The normalized results from all four anchors are then combined into one data set and overlain as shown in Figure 8.15. The envelopes of N*-w* indicate similar trends to those observed in Figure 8.10 and Figure 8.11, namely that the individual anchor demonstrates a trend of decreasing N* with increasing w*, however the envelope of all data shows less dependency between N*-w*. 
Figure 8.14 $N^*\text{-}w^*$ pair distribution by anchor for test V2.WALLE.F.N9.CC08.FM02.20P, envelope of data shown.
8.5.6 Load cutoff

It is observed that during oscillation of the component, a substantial number of near zero \( N \) values occur at low levels. Very low \( N \) values will have little to no impact on the behavior of the anchor. Therefore, these low levels bias the statistical summary of \( N-w \) correlation as the cracks may still be oscillating while the anchor is lightly loaded. In this section, we consider the selection of a low load cutoff to remove small loads from the statistical analysis, which are considered as too low to be considered relevant to the anchors’ performance, when compared with the ultimate tensile strength of the anchor. The load cutoff is herein expressed as a percentage of the mean ultimate strength of the anchor in cracked concrete \( N_{um,cr} \), as determined experimentally from reference tests conducted by Watkins and Hutchinson (2010a & 2010b) using a static crack width of 0.8mm [0.03 inch]. Figure 8.16 shows an example of a 5% load
cutoff applied to a subset of data from test V2.WALLE.F.N9.CC08.FM02.20P. For anchor type N9, $N_{um,cr} = 6.046$ kips, therefore $5\% N_{um,cr} = 0.3023$ kips. Below this load cutoff, all data points are removed as shown. For this same test, Figure 8.17 depicts the $N^* - w^*$ relationship per anchor after a $5\% N_{um,cr}$ load cutoff has been applied. Unless otherwise noted, the default load cutoff is 0\%, in other words, all negative load washer data is removed. Negative (compression) load washer readings are fictitious, likely a result of sensor error.

![Diagram](image.png)

**Figure 8.16** Example of $5\% N_{um,cr}$ load cutoff applied for test V2.WALLE.F.N9.CC08.FM02.20P
8.6 Statistical Analysis of Test Data

To understand the correlation between anchor load and crack width, a statistical analysis of the data is performed. This analysis includes calculation of the arithmetic mean (μ), standard deviation (σ) and coefficient of variation (CV) of load per crack width bin, where CV is defined as $\sigma / \mu$. Bin sizes were discussed previously in Section 8.4. In addition, the coefficient of variation Adopting the post-processing strategies described in the previous section, in what follows the various data groupings are analyzed. First, an analysis of the sensitivity of a key parameter, namely the load cutoff is undertaken using all data available. Secondly, the sensitivity of $N_i$ versus $N_{\text{max,k}}$ is studied. Thirdly, the sensitivity to data sampling rate is reviewed and finally the sensitivity of the data during crack opening versus crack closing is studied. However, before
describing the detailed analysis of $N$-$w$ correlation an overview of the correlation test input and response parameters is presented.

### 8.6.1 Overview of peak input and response parameters

The following section provides an overview of the correlation test motion peak input parameters as well as peak WALLE response parameters. The peak input acceleration, PIA, measured at the slab at the base of WALLE ranged from approximately 0.2 to 1.1g (Figure 8.18). The peak WALLE acceleration, PWA, response, as measured at the slab at the base of WALLE, ranged from approximately 0.25 to 2.8g. The component acceleration amplification, as measured by PWA/PIA ranged from 0.6 to 3.2 (Figure 8.19) demonstrating a level of component amplification that is consistent with current US design codes which specify a component amplification ratio, $a_p$, of 1.0 for stiff and $a_p$ of 2.5 for flexible components (ASCE 7, 2010).
Figure 8.18 Peak WALLE acceleration, PWA versus peak input acceleration, PIA

Figure 8.19 WALLE component amplification (PWA/PIA)

Figure 8.20 compares the PIA and PWA from the correlation and failure tests. The failure tests generally required higher PIA in order to fail the anchors. It is noted that the PWA for the
stiff WALLE correlation tests were higher than the failure tests because the WALLE weight was less 806 lbs versus 2550 lbs for the stiff and flexible WALLE respectively. Another means to evaluate the response is to compare to the spectral acceleration of the input motion at the period of WALLE.

Figure 8.20 Comparison of correlation and failure test PIA and PWA

Figure 8.21 describes the relationship between PWA and the input motion spectral acceleration calculated at the WALLE initial period, TWALLE, for 2% damping. In linear elastic system these should show a one-to-one relationship; however for the WALLE the PWA is typically always less than the elastic spectral acceleration. Contributing factors include; the anchored WALLE is a non-linear system that shifts period as the anchors are loaded, begin to displace, and cause rocking of the WALLE, the actual damping ratio ranged from 0.1 to 7% for the correlation tests. Moreover, the reference motion FM02 is highly peaked around the flexible
WALLE period of 0.25 seconds, therefore even a slight period elongation shifts the WALLE off the peak of the spectrum and greatly reduces the spectral demand.

During the correlation tests the drift ratio ranged from approximately 0.1% to 1.2% (Figure 8.22). The drift ratio was calculated as the ratio of the displacement at the center of mass over the height of the center of mass. The drift ratio increased with increasing peak input acceleration. The stiff WALLE experienced less drift at a given PIA because its weight was much less than the flexible WALLE.
8.6.2 Overview of \textit{N-w} data

The following section provides an overview of all \textit{N-w} data from the following tests: (1) variable phase correlation tests, version 1 and version 2 using undercut anchors, (2) failure test series using epoxy, expansion, drop-in, and undercut anchors, and (3) in-phase correlation tests using undercut anchors. The anchor load \(N\) is shown as a percentage of \(N_{\text{um,cr}}\) versus crack width in units of millimeters for the variable phase correlation tests, motions FM01 to FM20 for version 1 and version 2 test series for the flexible and stiff WALLE test series respectively (Figure 8.23 and Figure 8.24). These figure provides a sense for the physical magnitude of load imposed on the anchors during the tests with respect to their monotonic mean ultimate capacity in cracked concrete with 0.8mm constant crack width. It is noted that for many of the in-phase correlation tests, the anchor loads were intentionally kept low, as the maximum anchor load was targeted
between 10 and 50% \( N_{um,cr} \). Figure 8.23 also demonstrates that reasonable consistency is observed between the Version 1 and Version 2 test series.

Figure 8.25 and Figure 8.26 replot the correlation test data, now shown as \( N^* \) and \( w^* \). Normalization in this manner allows for comparison between tests conducted at different scale factors. It is noted that that a load cutoff of 5% \( N_{um,cr} \) has been applied in Figure 8.25 and Figure 8.26. Figure 8.27 summarizes \( \%N_{um,cr} - w \) data from all correlation tests. This figure demonstrates that the achieved maximum anchor loads varied from 10 to 80% \( N_{um,cr} \). In addition, although the maximum crack width was targeted for 0.8mm, the achieved maximum crack width was approximately 0.9mm. For comparison with the lower amplitude variable phase correlation tests, Figure 8.28 summarizes \( \%N_{um,cr} - w \) data from all failure tests. The achieved maximum anchor loads during the failure tests range from 125% \( N_{um,cr} \) (\( w_{max} = 0.8 \) mm) to 175% \( N_{um,cr} \) (\( w_{max} = 0.0 \) mm). The maximum crack width was targeted for 0.8 mm, while the achieved maximum crack width was approximately 1.0 mm. As noted previously, crack width overshoot was typically due to additional expansion forces at the anchor head when the anchor was highly loaded.
Figure 8.23 \( \%N_{um,cr} \cdot w \) for variable phase correlation tests, versions 1 and 2, FM01 to FM20, 5\( \%N_{um,cr} \) load cutoff (flexible WALLE), data overlaid for all four anchors
Figure 8.24 $\%N_{am,cr}-w$ for variable phase correlation tests, versions 1 and 2, FM01 to FM20, 5$\%N_{am,cr}$ load cutoff (stiff WALLE), data overlaid for all four anchors
Figure 8.25 $N^*-w^*$ for variable phase correlation tests, versions 1 and 2, FM01 to FM20, 5\%$N_{um,cr}$ load cutoff (flexible WALLE), data overlaid for all four anchors
Figure 8.26 $N^*-w^*$ for variable phase correlation tests, versions 1 and 2, FM01 to FM20, $5\%N_{max,cr}$ load cutoff (stiff WALLE), data overlaid for all four anchors
Figure 8.27 \( \% N_{um,cr}\cdot w \) all variable phase correlation tests overlaid with mean and one standard deviation, \( 5\% N_{um,cr} \) load cutoff

Figure 8.28 \( \% N_{um,cr}\cdot w \) all failure tests overlaid with mean and one standard deviation, \( 5\% N_{um,cr} \) load cutoff
8.6.3 Sensitivity of $N^*\cdot w^*$ considering low amplitude load cutoff

As shown in Figure 8.29, there are a large number of $N^*\cdot w^*$ pairs that occur in the very low load range (small $N^*$ values). The low load values may occur during the quiescent period before and after a test as well as in between load cycles. To investigate the sensitivity of results to load cutoff selection, a study was undertaken using 0%, 1%, 5%, 8%, 10% and 25% $N_{um,cr}$. The sensitivity study was conducted on all data $N_i$ as well as the maximum load per load cycle $N_{max,k}$. An example of a 5% $N_{um,cr}$ load cutoff is shown in Figure 8.30, with loads normalized by $N_{j,max}$. It is noted that the $N^*$ values omitted may be greater than 0.05$N^*$ if the maximum load achieved by the anchor are less than 100% $N_{um,cr}$. This is evident by the removal of loads up to 0.1$N^*$ in Figure 8.30.

![Figure 8.29 %$N_{um,cr}$-$w^*$ pairs with load normalized by $N_{um,cr}$ for test V2.WALLE.F.N9.CC08.FM02.20P](image-url)
Figure 8.30 \(N^*-w^*\) pairs with load normalized by \(N_{j,max}\) (5% \(N_{um,cr}\) cutoff) for test V2.WALLE.F.N9.CC08.FM02.20P

The average normalized load \(N^*_i\) in each crack width bin, using the various trial load cutoffs, 0%, 1%, 5%, 8%, 10% and 25% \(N_{um,cr}\) is shown in Figure 8.31 (data points are omitted for clarity and only the average load is shown). Part (a) of this plot presents data from the failure test series, Group G3 (M = 27, where M equals the number of tests), whereas part (b) shows data from only the correlation test series, Group G2 (M=41) where test groups are defined in Table 8.3. Figure 8.31 shows that the average load increases as the normalized cutoff load is increased (i.e. more data is removed from the lower load range thereby increasing the average load). It is noted that the normalized load cutoff is not sensitive to crack width (i.e. the load increases proportionately over all crack width bins). Figure 8.32 shows the average load based on the maximum load per load cycle \(N_{max,i}\) in each crack width bin. As compared to Figure 8.31, Figure 8.32 shows overall larger average loads and for the failure test dataset (Figure 8.32a) a more pronounced increasing \(N^*\) with increasing \(w^*\). As discussed later, this is likely due to the
fact that as the anchors are loaded with larger load amplitudes (greater than 50% \( N_{um,cr} \)) the expansion action of the anchors tends to force the cracks open. The mean \( N^*-w^* \) curves in Figure 8.31 and Figure 8.32 are generally flat, therefore, they demonstrate that anchor load and corresponding crack width are in general independent. Additionally, for any given normalized load cutoff there is still little dependence between \( N^* \) and \( w^* \). The exception to this is the 0% and 10% load cutoff case of Figure 8.31a and Figure 8.32a and the 25% \( N_{um,cr} \) load cutoff case of Figure 8.32b. In the former, the range of \( N^*_{max,k} \) is 0.05 to 0.25, demonstrating a fivefold increase as \( w^* \) varies from 0 to 1.0. This is most likely caused by the physical explanation given earlier that large load amplitudes tend to force the crack apart due to anchor expansion forces. In the later case (load cutoff of 25% \( N_{um,cr} \)), the \( N^*-w^* \) relationship is more sporadic and lacks a clear trend. The variability of \( N^*-w^* \) in this later case is likely due to the smaller data set of loads greater than 25% \( N_{um,cr} \). Recall the correlation test series targeted maximum anchors loads up to 50% \( N_{um,cr} \) whereas the failure test series targeted maximum anchor loads in excess of 100% \( N_{um,cr} \) in order to fail the anchors in tension. The average \( N^* \) ranges from approximately 0.1 to 0.7 with values of 0.1, 0.15, 0.4, 0.45, 0.5, and 0.7 for normalized load cutoffs of 0%, 1%, 5%, 8%, 10% and 25% \( N_{um,cr} \) respectively.

**Figure 8.31** Average normalized load: a) all failure tests and b) all variable phase correlation tests
The independence of anchor load and crack width means that it is reasonable to use the average load over all crack width bins as an indicator of the average load under the influence of a crack width, which cycles from 0 to 1.0 $w_{\text{max}}$. As such, Figure 8.33 shows the average normalized load and average normalized load plus or minus one standard deviation obtained over all crack width bins using the various cutoffs and the selected test groupings (e.g. all correlation, and flexible / stiff WALLE). These data articulate the relative consistency between results of the stiff and flexible WALLE configurations, indicating that NCS period does not affect the average normalized load across all crack width bins. One may also observe that with increasing load cutoff, the average normalized load begins to stabilize. This saturation effect is articulated in Figure 8.34. The plot shows a fourfold increase in $N^*$ as load cutoff increases from 0% to 5% whereas there is less than a two fold increase in $N^*$ as load cutoff increases from 5% to 25%.
Based on a review of all test data, a cutoff of 5% $N_{um,cr}$, is selected as an acceptable cutoff and is used for the remainder of the analyses. It is noted from Figure 8.34 that load cutoffs of 5, 8, and 10% $N_{um,cr}$ demonstrate a convergence in the $N^*$ amplitudes, in this sense selection of 5% $N_{um,cr}$ yields conservatism in the selection of $N^*$. The selection of 25% $N_{um,cr}$ as a load cutoff is not deemed feasible as it encroaches on removing valuable load levels that affect anchor performance. This is confirmed by observing the randomness in response of the $N^*-w^*$ trend of Figure 8.32b for the 25% $N_{um,cr}$ load cutoff case.
8.6.4 Sensitivity of load data versus maximum load per cycle data ($N^*_i$ vs. $N^*_{max,k}$)

In this section, we study the correlation between $N$ and $w$, by considering all load data as well as the maximum loads per load cycles ($N^*_i$ versus $N^*_{max,k}$). In addition, we consider more carefully the dispersion in the data. Considering one standard deviation allows the user flexibility in choosing an appropriate anchor load level commensurate with the desired level of safety. Figure 8.35 and Figure 8.36 summarize the average normalized anchor load and one standard deviation envelope for the correlation and failure tests, respectively using a 5% $N_{um,cr}$ load cutoff. The one standard deviation envelope ranges from approximately 0.6 to 0.8 $N^*$ considering both $N^*_i$ and $N^*_{max,k}$. 

Figure 8.34 Effect of load cutoff on normalized load $N^*$ averaged over all $w^*$ bins

![Graph showing the effect of load cutoff on normalized load $N^*$ averaged over all $w^*$ bins.](image)
Figure 8.35 Average normalized anchor load and one standard deviation envelope (5\% N_{um,cr} cutoff), all variable phase correlation tests

Figure 8.36 Average normalized anchor load and one standard deviation envelope (5\% N_{um,cr} cutoff), all failure tests
The effect of considering all data \((N_i)\) versus considering maximum loads per cycle \((N_{\text{max},k})\) is demonstrated in Figure 8.37. The average normalized load is approximately 0.4 consistently across all normalized crack width bins. These summary analyses further demonstrate the lack of dependency of \(N^*\) on \(w^*\) and further support the contention that a constant amplitude load during a crack cycling anchor test will on average be able to capture its load-deformation characteristics. It is also noted that the average normalized load is approximately the same for the correlation tests and the failure tests, substantiating selection of a constant value of \(N^* = 0.4\), over a broad range of expected axial tension load demands \((N > 5\% N_{\text{um,cr}}\text{ up to failure})\).

![Graph showing average normalized load for all failure tests and all variable phase correlation tests (5\% \(N_{\text{um,cr}}\) cutoff)](image)

**Figure 8.37** Average normalized load for all failure tests and all variable phase correlation tests (5\% \(N_{\text{um,cr}}\) cutoff)

Table 8.5 summarizes the average and average plus one standard deviation \(N^*-w^*\) results for all failure and all correlation tests for both 0\% and 5\% \(N_{\text{um,cr}}\) load cutoffs, all data \((N_i)\), and load cycle peaks \((N_{\text{max},k})\). Equivalent weight is given to each \(w^*\) bin. Due to the independency of
The results are not affected by considering a weighting of the $w^*$ bins. This table provides the interested reader with all of the information needed to select a constant load or varying load coupled with crack width bin, should this be needed for crack cycling testing.

Table 8.5 Normalized load and crack width summary statistics

<table>
<thead>
<tr>
<th>Load cutoff: 0% $\numcr$</th>
<th>Normalized Crack Width $w^*$</th>
<th>0.1</th>
<th>0.2</th>
<th>0.3</th>
<th>0.4</th>
<th>0.5</th>
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<td>0.10</td>
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Load cutoff: 5% $\numcr$

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<th>Normalized Crack Width $w^*$</th>
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<td>0.33</td>
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<tr>
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</table>

### 8.6.5 Effect of Sampling

All test data are sampled at 200 samples per second. Although this is deemed more than sufficient to capture the component behavior, which oscillated at or around its natural frequency of 4 to 10Hz; and the crack width histories, which oscillated between 0.45 to 4Hz. However, to confirm this, we study the sensitivity of the resulting $N-w$ correlations to sampling rate. To characterize the sensitivity, we analyze the mean load per crack width bin for the following sampling rates: 200Hz, 100Hz, 50Hz and 10Hz. The effect of varying the sampling rate may be demonstrated using a load cutoff of 5% $\numcr$; however, to include the largest data set possible,
no load cutoff was applied. Figure 8.38 demonstrates that rate of data sampling has negligible effect on the mean $N^*-w^*$ statistics. For finely sampled data, i.e. rates between 50 and 200 Hz, the normalized load is constantly around 0.1. It is recommended to use a sampling rate greater than or equal to the Nyquist frequency to avoid any sampling rate effects.

Figure 8.38 Effect of data sampling rate on $N^*$ for all crack width bins

8.6.6 Effect of Crack Opening and Closing

The following analysis studies the $N-w$ behavior that occurs during the opening, closing and constant stages of crack cycling. Figure 8.39 displays the crack width history for the Northwest anchor crack during test V2.WALLE.F.N9.CC08.FM02.20P with the opening, closing and constant stages of crack cycling identified. A windowed moving average slope algorithm was applied over a sliding window of 1/25 seconds with a crack slope cutoff of +/- 0.25 mm/sec as the threshold between an opening/closing crack and a constant crack slope. Figure 8.40 shows the same crack history over the windowed region marked in Figure 8.39. The stages of crack cycling are identified as opening, closing and relatively constant. The $N^*-w^*$ data from all correlation
tests (Figure 8.41) and all failure tests (Figure 8.42) were reprocessed and the $N^*-w^*$ pairs that occur during the three stages of crack cycling were separated. Similar trends were observed as in the case for all data, namely that there is a general lack of dependency between $N^*$ and $w^*$. In addition, the average normalized load hovers around 0.3 to 0.4, for each crack configuration consistent with prior results.

![Figure 8.39 Crack opening, closing, and constant stages identified for the crack adjacent to the Northwest anchor during test V2.WALLE.F.N9.CC08.FM02.20P](image)

Figure 8.39 Crack opening, closing, and constant stages identified for the crack adjacent to the Northwest anchor during test V2.WALLE.F.N9.CC08.FM02.20P
Figure 8.40 Windowed portion of test V2.WALLE.F.N9.CC08.FM02.20P demonstrating crack opening, closing, and constant stages identified for the crack adjacent to the Northwest anchor.
Figure 8.41 $N^*-w^*$ relationships for crack opening, closing, and constant stages for all correlation tests

Figure 8.42 $N^*-w^*$ relationships for crack opening, closing, and constant stages for all failure tests
8.7 In-Phase Correlation Tests

The goal of the $N^*-w^*$ analysis was to identify if trends exist between anchor axial load and crack width. The prior analyses used the results of variable phase correlation tests (i.e. natural phasing that occurs between anchor loading and the crack cycling of the building component). Tests were also conducted using anchor loading and crack cycling that were targeted to be in-phase. In what follows we describe the test protocol and results of this portion of the study.

8.7.1 In-phase test results and interpretation

Before presenting results of the in-phase correlation tests it is instructive to describe the different types of trends in $N^*-w^*$ that may be observed. A horizontal line in the $N^*-w^*$ plots indicates that $N^*$ is uncorrelated with $w^*$. A positive sloped line in the $N^*-w^*$ plots indicates in-phase behavior (large load concurrent with large crack width). A negative sloped line in the $N^*-w^*$ plots indicates out-of-phase behavior (small load concurrent with large crack width).

The in-phase test setup and protocol was described in detail in an earlier chapter on test setup. This section describes the results from the two motions used during in-phase testing, namely: (a) load and crack protocol CP1, Wood et al. (2009) and (b) the crack history from earthquake FM14, a motion from the numerical study of Wood et al. (2009), taken from the 5th floor of a 12 story building subjected to the Superstition Hills ground motion. The resulting in-phase anchor tension load and crack width histories are shown in Figure 8.43 for in-phase crack cycling protocol (CP1) and Figure 8.44 for the target in-phase FM14 earthquake motion. For clarity, $w$ is plotted as the average of two crack width measurements of a pair of anchors on the North or South side respectively. It is noted that while perfect in-phase correlation between $N-w$ was targeted, in each test, one pair of anchors did not achieve the target in-phase relationship due
to a nominal lag in experimental system performance. For example, in Figure 8.43 the South pair of anchors had a resulting out-of-phase $N$-$w$ relationship and in Figure 8.44 the North pair of anchor had an out-of-phase relationship between $N$ and $w$. However, the opposite pair of anchors did achieve the targeted in-phase correlation desired for this study. To further visualize the $N$-$w$ correlation, the maximum anchor loads ($N_{\text{max}}$) are identified along with the corresponding crack width at the instance of maximum load ($w@N_{\text{max}}$). It is noted that in Figure 8.43 (South anchors) and Figure 8.44 (North anchors), that nearly perfect out-of-phase behavior results in $w \rightarrow 0$ at $N_{\text{max}}$, whereas for the opposing pair of anchors $w \rightarrow w_{\text{max}}$ at $N_{\text{max}}$.

![Diagram showing the N-w correlation]

**Figure 8.43** $N$-$w$ history due to in-phase crack cycling protocol CP1 Wood et al. (2009) (STRUCT.N9.CC08.CP2.D1c)
Figure 8.45 depicts a windowed in portion of the \( N \) and \( w \) histories for CP1 showing that the North-West (NW) and North-East (NE) anchors were responding in-phase with crack width cycling, whereas the South-West (SW) and South-East (SE) anchor loads were responding out-of-phase with crack width cycling. Likewise, real out-of-phasing is observed to the North anchor pair when implementing the FM14 motion (Figure 8.46). Nevertheless, the perfect phasing achieved in the pairs if each test series is instructive to illustrate extreme idealized conditions imposed upon the anchor.
Figure 8.45 $N$-$w$ history (zoomed in) due to target in-phase crack cycling protocol, CP1

Wood et al. (2009) (STRUCT.N9.CC08.CP2.D1c)

Figure 8.46 $N$-$w$ history (zoomed in) due to target in-phase FM14 motion

(STRUCT.N9.CC08.CR14.D14c)

Synthesizing all in phase test protocol CP1 data, Figure 8.47 and Figure 8.48 further demonstrate the impact of phasing on the $N^*-w^*$ behavior. Anchors SW and SE demonstrate out of phase behavior whereas anchor NW and NE demonstrate in phase behavior under the target in phase test CP1. Incidentally, these figures also demonstrate how difficult it is to obtain correlated
behavior even in a controlled laboratory setting. Anchors NW and NE were also intended to have negative correlated behavior; however, due to the test setup and movement of the slab due to the cracking actuators, the opposite effect occurred. Under extreme conditions as shown in Figure 8.47 (SE & SW anchors) perfect phasing can result in a dependency of $N^*$ inversely proportional to $w^*$.

Figure 8.47 Negative and positive correlated $N^*-w^*$ behavior target in-phase CP1 test:

STRUCT.N9.CC08.CP2.D1c
Figure 8.48 $N^*_{max,k}$-$w^*$ behavior for target in-phase CP1 protocol test:

STRUCT.N9.CC08.CP2.D1c

Synthesizing all in phase test earthquake FM14 data, Figure 8.49 and Figure 8.50 further demonstrate the impact of phasing on the $N^*-$w* behavior. Anchors NW and NE demonstrate out of phase behavior whereas anchor SW and SE demonstrate in phase behavior under the target in phase test FM14.
Figure 8.49 Negative and positive correlated $N^*-w^*$ behavior for target in-phase FM14 test:

STRUCT.N9.CC08.CR14.D14c

Figure 8.50 $N_{\text{max},k}^*-w^*$ behavior for target in-phase FM14 test:

STRUCT.N9.CC08.CR14.D14c
8.8 Period of Crack Cycling and NCS Anchor Loading (Period Ratio)

The following section presents the data in a systematic way to examine the effect of varying period of anchor loading ($T_{p,N}$) and the period of crack cycling ($T_{p,cr}$) on the $N$-$w$ correlation. It is noted that the periods are defined for one complete sine cycle for both loading and cracking. In the extreme case when $T_{p,N}$ equals $T_{p,cr}$ it is unclear if this causes $N$-$w$ to be more highly in phase. For the purpose of this investigation, the period ratio of the predominant period of crack cycling ($T_{p,cr}$) to the predominant period of anchor loading ($T_{p,N}$) is defined as ($T_p^*$), i.e.:

$$\text{Period Ratio } = T_p^* = \frac{T_{p,cr}}{T_{p,N}}$$

A period ratio of 1.0 indicates that the cracking history oscillates at the same rate as the anchor loading history. This is hypothesized to be the most likely condition to cause maximum anchor load to be correlated with maximum crack width. Also integer multiples of 1.0 (harmonics in period ratio) are hypothesized to potentially cause highly correlated behavior, i.e. where the crack cycling period rate is double or triple the anchor load cycling rate. It is noted that a ratio of 1.0 does not guarantee negative correlated behavior (large load with large crack width) because the signals could be out of phase with each other. For example, for harmonic crack and load cycling, a 90 degrees phase lag with a period ratio of 1.0 would cause out-of-phase behavior (large load with small crack width), whereas a period ratio of 1.0 with a 0 degree phase lag could cause perfect in-phase $N$-$w$ behavior.

Numerical building simulations with varying dynamic characteristics were used as input to the variable phase correlation tests. Using the correlation test results, it is possible to study the effect of the building dynamics on the correlation between anchor loading and crack cycling. As
discussed previously, the predominant period of cracking $T_{p,cr}$ may be directly related to the first or second natural periods of the building ($T_1$ and $T_2$). Table 8.6 shows the elastic natural modal periods of the buildings determined from Eigenvalue analysis (Wood et al., 2009) and the average predominant period of cracking from the building time history simulations. The first mode period is noted to be highly correlated with the predominant period of cracking as indicated by period ratios ($T_{p,cr}/T_1$) very close to 1.0. The ratio is slightly greater than unity due to nonlinear action of the building.

Table 8.6 Average predominant period of cracking ($T_{p,cr}$) and comparison with elastic modal periods of building models based on simulation results of Wood et al. (2009)

<table>
<thead>
<tr>
<th>Parameter of Interest</th>
<th>2 story</th>
<th>4 story</th>
<th>8 story</th>
<th>12 story</th>
<th>20 story</th>
</tr>
</thead>
<tbody>
<tr>
<td>$T_{p,cr}$ (sec)</td>
<td>0.26</td>
<td>0.49</td>
<td>0.97</td>
<td>1.43</td>
<td>2.03</td>
</tr>
<tr>
<td>$T_1$ (sec)</td>
<td>0.24</td>
<td>0.47</td>
<td>0.89</td>
<td>1.33</td>
<td>2.07</td>
</tr>
<tr>
<td>$T_2$ (sec)</td>
<td>0.06</td>
<td>0.13</td>
<td>0.29</td>
<td>0.45</td>
<td>0.71</td>
</tr>
<tr>
<td>$T_3$ (sec)</td>
<td>0.02</td>
<td>0.07</td>
<td>0.15</td>
<td>0.24</td>
<td>0.39</td>
</tr>
<tr>
<td>$T_4$ (sec)</td>
<td>0.02</td>
<td>0.04</td>
<td>0.1</td>
<td>0.16</td>
<td>0.26</td>
</tr>
<tr>
<td>$T_{p,cr}/T_1$</td>
<td>1.08</td>
<td>1.05</td>
<td>1.09</td>
<td>1.07</td>
<td>0.98</td>
</tr>
</tbody>
</table>

Table 8.7 summarizes the period values of interest, and in particular provides the predominant period of anchor loading ($T_{p,N}$), predominant period of cracking ($T_{p,cr}$), and period ratio ($T_p^*$) measured experimentally and from simulations. The period ratio ranges from 1.0 (potentially in phase loading and crack cycling) to approximately 17 (potentially out of phase anchor loading and crack cycling). Table 8.7 values indicate that the period ratios increase with increasing building height. In addition, they are smaller for the flexible WALLE configuration. These trends are expected due to the generally lower period characteristics of NCS as compared with buildings. One would expect that the likely case for in-phase behavior is for the 2 story building for flexible WALLE and stiff WALLE, where the period ratios are 1.0 and 2.0 respectively. However, as shown in Figure 8.52 and Figure 8.51, the 2 story building shows independence of the average $N^*-w^*$ behavior, similar to the other building heights.
Table 8.7 Experimental and simulation period values of interest

<table>
<thead>
<tr>
<th>Parameter Description</th>
<th>Parameter of Interest</th>
<th>2 st</th>
<th>4 st</th>
<th>8 st</th>
<th>12 st</th>
<th>20 st</th>
</tr>
</thead>
<tbody>
<tr>
<td>Predominant period of cracking from simulation (Wood et al. 2009)</td>
<td>$T_{p,cr,sim}$</td>
<td>0.26</td>
<td>0.49</td>
<td>0.97</td>
<td>1.43</td>
<td>2.03</td>
</tr>
<tr>
<td>Elastic first mode building period (Wood et al. 2009)</td>
<td>$T_f$</td>
<td>0.24</td>
<td>0.47</td>
<td>0.89</td>
<td>1.33</td>
<td>2.07</td>
</tr>
<tr>
<td>Theoretical WALLE period (stiff configuration)</td>
<td>$T_{n,walle, stiff}$</td>
<td>0.10</td>
<td>0.10</td>
<td>0.10</td>
<td>0.10</td>
<td>0.10</td>
</tr>
<tr>
<td>Theoretical WALLE period (flexible configuration)</td>
<td>$T_{n,walle,flex}$</td>
<td>0.25</td>
<td>0.25</td>
<td>0.25</td>
<td>0.25</td>
<td>0.25</td>
</tr>
<tr>
<td>Experimental WALLE period (stiff, from correlation tests, from PSD transfer function)</td>
<td>$T_{p,WALLE,exp, stiff}$</td>
<td>0.13</td>
<td>0.12</td>
<td>0.12</td>
<td>0.12</td>
<td>0.12</td>
</tr>
<tr>
<td>Experimental WALLE period (flexible, from correlation tests, from PSD transfer function)</td>
<td>$T_{p,WALLE,exp, flex}$</td>
<td>0.27</td>
<td>0.27</td>
<td>0.26</td>
<td>0.26</td>
<td>0.25</td>
</tr>
<tr>
<td>Experimental WALLE period (stiff, from anchor loading, correlation tests)</td>
<td>$T_{p,N, stiff}$</td>
<td>0.13</td>
<td>0.12</td>
<td>0.12</td>
<td>0.13</td>
<td>0.12</td>
</tr>
<tr>
<td>Experimental WALLE period (flex, from anchor loading, correlation tests)</td>
<td>$T_{p,N, flex}$</td>
<td>0.26</td>
<td>0.26</td>
<td>0.26</td>
<td>0.26</td>
<td>0.26</td>
</tr>
<tr>
<td>Period ratio, stiff WALLE ($T^*_{p, stiff}$)</td>
<td>$T_{p,cr,sim}/T_{p,N, stiff}$</td>
<td>2.0</td>
<td>4.1</td>
<td>8.1</td>
<td>11.0</td>
<td>16.9</td>
</tr>
<tr>
<td>Period ratio, flexible WALLE ($T^*_{p, flex}$)</td>
<td>$T_{p,cr,sim}/T_{p,N, flex}$</td>
<td>1.0</td>
<td>1.9</td>
<td>3.7</td>
<td>5.5</td>
<td>7.8</td>
</tr>
</tbody>
</table>
Figure 8.51 Effect of building height on $N^*-w^*$ behavior (flexible WALLE) (a) with 0% $N_{um,cr}$ load cutoff and (b) with 5% $N_{um,cr}$ load cutoff.

Figure 8.52 Effect of building height on $N^*-w^*$ behavior (stiff WALLE) (a) with 0% $N_{um,cr}$ load cutoff applied and (b) with 5% $N_{um,cr}$ load cutoff applied.

A graphical summary of the $N^*-w^*$ relationship for all test groups is provided in Figure 8.53, for the correlation tests using a 5% $N_{um,cr}$ load cutoff applied. These data are now separated into test groups associated with the different building heights and WALLE periods. The flatness of the $N^*-w^*$ traces highlights the lack of dependence between $N^*$ and $w^*$. Figure 8.53 also plots the $N^*-w^*$ results using all data $N_i$ and only load cycle peaks $N_{max,k}$. In all cases the trends are similar with the $N_{max,k}$ values being slightly larger.
Figure 8.53 Summary of $N^*-w^*$ relationships for variable phase correlation test groups considering different building and WALLE configurations (Note: $M$ = number of tests = 4 for each subplot)
8.9 Effect of Anchor Type

Figure 8.54 describes the effect of anchor type on the $N^*-w^*$ relationship. There is a slight increasing trend that as anchor load increase as crack width increases. Review of the test data indicates that this is because during the testing, the expansion forces from the anchor holding mechanism tend to expand the crack further apart when the anchor is highly loaded.

![Graph showing effect of anchor type on $N^*-w^*$ relationship](image)

A graphical summary of the $N^*-w^*$ relationship for all test groups is provided in Figure 8.53, for the correlation tests using a 5% $N_{am,cr}$ load cutoff applied. These data are separated into test groups associated with the different building heights and WALLE periods. The flatness of the $N^*-w^*$ traces highlights the lack of dependence between $N^*$ and $w^*$. One exception to this are the in-phase correlation “STRUCT” tests where $N^*-w^*$ dependence was forced by the prescribed loading and crack cycling conditions. The other exception is in the failure tests, especially the expansion anchor test groups “SUM.N5.CC05 and SUM.N5.CC08”, where a large load on the anchor causes expansion forces that wedge the crack apart, resulting in a slight dependence between $N^*$ and $w^*$. Figure 8.53 also plots the $N^*-w^*$ results using all data $Ni$ and only load cycle peaks $N_{max,k}$. In all cases the trends are similar with the $N_{max,k}$ values being slightly larger.
Figure 8.55 Summary of $N^* - w^*$ relationships for failure test groups considering different anchor types and constant period ratio, $T^*_p = 1.0$ ($M =$ number of tests)
8.10 Summary Remarks

Based on the $N^*-w^*$ analysis, the following summary remarks are made:

- Anchor load and crack width were found to be uncorrelated when the inertial loading is randomly (naturally) phased with the crack cyclic history. In other words, for variable period NCSs and random earthquake motions, $N^*$ shows little dependence on $w^*$. Reasons for this include the variable nature of all of the factors that determine crack cycling in concrete flexural members such as ground motion characteristics, soil parameters, building dynamic properties as well as the variable nature of anchor loading such as NCS dynamic properties, characteristics of the floor input motion as determined by the building dynamic properties and which floor level the NCS is located on. This further supports the contention that a constant amplitude load during a crack cycling anchor test will sufficiently capture its load-deformation characteristics.

- A load cutoff of 5% $N_{\text{cr,cr}}$, is considered reasonable to meet the goal of removing low loads that do not affect anchor performance, because low loads may skew the statistics as cracks continue to oscillate when the component is not oscillating.

- The independence between $N^*$ on $w^*$ was further substantiated by analyses that considered anchor loading during the following three crack cases: 1) cracks opening, 2) cracks closing, and 3) cracks approximately constant. All three case shown little dependency between $N^*$ and $w^*$.

- An average value of $\alpha_{cc} = 0.4$ independent of crack width is proposed to capture the reduction in anchor tensile load under seismic demands. If one wishes to consider plus one standard deviation then conservatively use $\alpha_{cc} = 0.65$. Both may be applied
to a constant load during crack cycling due to the lack of dependency observed between $N^*$ and $w^*$.

- The average normalized load, $N^*$, was not sensitive to the scale of the test (i.e. low to moderate amplitude loading level correlation tests versus high amplitude failure tests). However, the $N^* - w^*$ experimental correlations have a slight increasing trend for the failure test series, because large anchor loads tend to wedge the crack apart for some types of anchors that achieve their fastening mechanism by expansion against the drilled hole.

- The results are independent of sampling frequency as long as the sampling frequency is sufficiently fine as to capture the relevant load and crack cycling data. It is recommended to use at least twice the Nyquist frequency of the natural frequency of loading or crack cycling.

- The results of all load data $N_i$ and maximum load per load cycle $N_{\text{max},k}$ are similar with $N_{\text{max},k}$ being slightly larger. It is recognized however, that $N_{\text{max},k}$ data may be sufficient to characterize loads, which most significantly affect the performance of the anchors.

- It was found that the case where the period of anchor loading and period of crack cycling are equal (i.e. a period ratio of 1.0) did not produce correlated $N^* - w^*$ behavior, likely due to phase-lag between the two processes.

- During the in-phase correlation tests, it was possible to produce in-phase correlation between $N^* - w^*$; however, even under controlled laboratory conditions this behavior is not easily reproduced.
Chapter 9 Anchor Modeling

9.1 Introduction

This chapter presents the development of a nonlinear element that can be used to model anchor load-deflection ($N-\delta$) behavior in cracked and uncracked concrete for tension load cycling dominant applications. The nonlinear anchor model takes a complex physical mechanism at the micro level that involves concrete bearing, local concrete crushing, concrete cracking, friction between steel-to-steel elements, and friction between steel-to-concrete elements and simplifies the problem to the macro level of anchor load-displacement behavior. The anchor model is calibrated against single anchor tests and subsequently extended for use in a system model of the anchored-WALLE system to predict NCS system response.

Figure 9.1 provides an overview of the numerical modeling efforts. First, (Figure 9.1a) a local line element anchor macro model was developed that uses a tension-only hook element in series with a nonlinear element employing a pivot hysteresis rule (Dowell et al., 1998). The purpose of the hook element was to allow for tension-only anchor behavior and subsequently residual displacement modeling. The pivot hysteresis rule specific to an anchor type/failure mechanism is calibrated using available test data. The line element was implemented using SAP2000 software, therefore it could be implemented into other commercially structural analysis software. The anchor model may be used to predict anchor tension load, $N$, and anchor axial displacement, $\delta$, quantities. Secondly, (Figure 9.1b) the nonlinear anchor element was implemented into a system model of the anchored-WALLE system with the goal of predicting system response quantities, such as, inertial system force, $F$, and displacement, $\Delta$, at the WALLE center of mass.
9.2 Load Cycling versus Crack Cycling Dominant Behavior

Before introducing the anchor model, it is useful to review load cycling and crack cycling behavior. Anchor behavior may be categorized as (i) load cycling dominated or (ii) crack cycling dominated based on the combination of anchor load cycling and crack cycling in-situ conditions that the anchor experiences. Definitions are provided below and are explained in more detail in the following sections:

Load cycling dominant behavior: the frequency of anchor load cycling is greater than the frequency of crack cycling. In other words, load cycling dominant behavior is observed when the anchor load cycles at least once while the crack is in a relatively
static state of being closed, open or partially open. As a result, several loading cycles will occur during a single crack cycle.

**Crack cycling dominant behavior:** the frequency of crack cycling is greater than the frequency of anchor load cycling. In other words, crack cycling dominant is when the crack cycles at least one or more times while there is a relatively constant load on the anchor. As a result, only a partial load cycle will have occurred, while many crack cycles occur.

Figure 9.2 (part 1), shows a floor mounted NCS which, experiences tension load cycling due to seismic load transmitted through the building frame and subsequently through the NCS. If the frequency of oscillation of the NCS is higher than the frequency of crack cycling, then the anchorage will have tension load cycling under relatively constant crack width or at least less than one cycle of cracking. This can be said to be load cycling dominated. It is noted that for suspended NCSs (Figure 9.2, part 2) or certain wall mounted components, constant loads are imposed due to the self-weight of the NCS. Therefore, the anchors may experience crack cycling under constant or pulsating loads. It remains to be determined if this later case will result in anchorage behavior that is crack cycling dominated.

The floor motions imparted on a floor mounted NCS on the other hand will be dictated largely by the response of the building. The dynamic characteristics of the NCS in turn then dictate the frequency of the cyclic load demand imparted to the anchors. For the range of buildings considered by this study in the correlation tests, 2 story to 20 story, the first natural frequency and hence crack cycling frequency ranged from 0.5 to 4 Hz and the WALLE frequencies considered were 4 to 10 Hz. These ranges were felt to encompass a broad range of building and NCS stock in the field. As a result, the WALLE anchorage will be tension load cycling
dominated. Therefore, the anchor modeling efforts were directed towards the load cycling dominated case.

Figure 9.2 Examples of NCSs subjected to (1) tension load cycling and (2) constant or pulsating tension load with crack cycling

9.2.1 Load Cycling Dominant Behavior

To develop a robust hysteresis model for anchor load cycling behavior, cases of tension load cycling from past research programs are examined. Hoehler (2006) demonstrated that load-cycling behavior follows the monotonic envelope when loads are cycled at or near ultimate load for pull-through (Figure 9.3) as well as for breakout failure (Figure 9.4). In both cases an expansion anchor was used, but with different embedment depths to force the different failure modes. The important characteristic is that load cycling follows the monotonic envelop on the plateau and descending branch after ultimate load has been achieved and the unloading stiffness is approximately vertical for the expansion anchor.
Figure 9.3 Load-displacement curve for an expansion anchor failing by concrete breakout for tension load cycling after ultimate load (Hoehler, 2006)

Figure 9.4 Load-displacement curve for an expansion anchor failing by concrete breakout for tension load cycling at ultimate load (Hoehler, 2006)

Figure 9.5 shows the results of load cycling behavior for an expansion anchor that failed by pull-through (Mahrenholtz, 2009a). Figure 9.6 shows the results of load cycling behavior for an undercut anchor that failed by concrete breakout (Mahrenholtz, 2009a). In both cases, it is observed that the load-displacement behavior follows the mean monotonic
backbone curve during load cycling. Another salient characteristic of the load-cycling curves is that the unloading stiffness tends to be approximately vertical for the expansion anchor and approximately equal to the initial stiffness for the undercut anchor.

Figure 9.5 Load-displacement curve for an expansion anchor failing by pull-through (Mahrenholtz, 2009a)
Figure 9.6 Load-displacement curve for an undercut anchor failing by concrete breakout
(Mahrenholtz, P. 2009)

9.2.2 Crack Cycling Dominant Behavior

Mahrenholtz (2009b) conducted tests to examine the effect of crack cycling under constant tension load equal to 40% of the monotonic reference test ultimate load in static cracks. Load-displacement behavior during crack cycling followed by a monotonic tension test are displayed for an expansion anchor failing by pull-through (Figure 9.7) and for an undercut anchor failing by concrete breakout (Figure 9.8). In both cases, increasing displacement under constant load during crack cycling is observed. This type of displacement behavior is sometimes referred to as *ratcheting* because the displacement increases incrementally more each time the crack is cycled open.
Figure 9.7 Load-displacement curve for an expansion anchor during crack cycling at constant load followed by monotonic tension to failure (Mahrenholtz, 2009b)

Figure 9.8 Load-displacement curve for an undercut anchor during crack cycling at constant load followed by monotonic tension to failure (Mahrenholtz, 2009b)
9.3 Development of Anchor Load-Displacement Model (N-δ)

A nonlinear element anchor model is developed to capture tension dominated cases where load cycling is the dominant condition on the anchors (i.e. shear load cycling is not considered, thus if one considers ACI 318, 2008 guidelines, the model is limited to anchorage where shear force is less than 20% of the anchor shear capacity).

9.3.1 Nonlinear Anchor Model

A tension-only hook element and a nonlinear link element are used in series to completely model the tension only nonlinear hysteresis of the anchor (Figure 9.9). Both are link elements and therefore the nonlinear properties are defined independent of the length of the element. The hook element uses a very large stiffness and an initial opening of 0 inches to model tension only behavior. The purpose of the hook element is to assure tension-only behavior of the anchor. The initial gap represents any gap that exists between the bottom of the nut and the top of the baseplate. The stiffness of the hook should be at least 10 times that of the anchor, but not more than 100 times in order to reduce disparities in the stiffness matrix, which can cause model instability. The nonlinear anchor element employs the Pivot hysteresis model of Dowell et al. (1998) using the complete monotonic backbone curve from tension reference testing. The parameters used in the pivot model are defined in the following section.
9.3.2 Pivot Hysteresis Model

A hysteretic model that points towards a common point or *pivot point* to model degrading unloading stiffness was originally proposed by Kunnath et al. (1990) and was more completely developed and extended for use with a softened elastic loading stiffness by Dowell et al. (1998). It has since been implemented into the Computers and Structures finite element analysis software, SAP2000 (CSI, 2010). One of the unique features of the pivot model is the ability of the user to specify the location of the pivot point for unloading and reloading stiffness. This is particularly useful for modeling anchor tension load cycling where the unloading and reloading stiffness are very steep, typically ranging between the anchor initial stiffness and infinity. The user defined parameters for the pivot model in SAP2000 are shown in Figure 9.10.
The pivot model parameters are fully documented in the SAP2000 analysis reference manual (CSI, 2010) and are summarized below as relevant for anchor modeling. For anchor tension-only behavior we are only concerned with the upper right quadrant where the anchor is in tension and the anchor displacement is positive when pulled out of the concrete (Figure 9.10, quadrant, Q1). Therefore, the input parameters of interest for the pivot model are as follows:

- **Effective stiffness** – The user enters the effective stiffness of the anchor to be used only for linear elastic analysis cases

- **Multi-linear Force Deformation Definition** – The user enters the $N-\delta$ points along the mean anchor backbone curve obtained from monotonic tension testing in concrete having a crack width that is appropriate for the anchor
model being considered (note: positive $N$ is tension). A sufficient number of points should be defined to approximate the full $N-\delta$ mean monotonic backbone curve including the descending branch. On the compression side of the $N-\delta$ curve an elastic compression stiffness need not be defined, as the tension-only hook element in series with the anchor element, will not allow compression resistance to be engaged.

- **Tension yield force**: $F_{y1}$ - SAP 2000 considers this as the first positive point defined in the force-displacement definition. In the case of anchors, $F_{y1}$, is used in combination with, $\delta_{y1}$, to define the initial elastic tension stiffness. For anchors that have preload, it can be used to define the load at which the preload is overcome and the anchor begins to displace.

- **Pivot Location Parameter**: $\alpha_1$ – scalar multiplier on $F_{y1}$ which locates the pivot point $P_1$ for unloading to zero from positive force (Figure 9.11). Anchor unloading stiffness remains constant for load cycling this parameter is set to a very large number so that there is no degradation in unloading stiffness. As $\alpha$ approaches infinity the unloading stiffness, $K_{ul}$, approaches the initial stiffness, $K_i$.

- The parameters $F_{y2}$, $\beta$, $\alpha_2$, and $\eta$ shown in the input screen of Figure 9.10 are not required for anchor modeling.
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Figure 9.11 Effect of the alpha parameter on the location of the pivot point

9.3.3 Implementation of Anchor Model

The anchor types/failure modes shown in Table 9.1 were considered for model development: (1) pull-through failure of an expansion anchor and (2) concrete breakout failure of an undercut anchor. These anchors types/failure mechanisms were chosen because of their common use in design practice as well as their displacement capacity after ultimate load has been achieved.

Table 9.1 Sample tests of hysteretic results

<table>
<thead>
<tr>
<th>Anchor Type</th>
<th>Failure Mode</th>
<th>Test Name</th>
<th>Anchor Location</th>
<th>Max. Crack Width</th>
</tr>
</thead>
<tbody>
<tr>
<td>Expansion</td>
<td>Pull-through</td>
<td>V2.WALLE.F.N5.CC05.100P</td>
<td>SouthWest</td>
<td>0.5 mm</td>
</tr>
<tr>
<td>Undercut</td>
<td>Breakout</td>
<td>V2.WALLE.F.N9.CC08.40P</td>
<td>SouthEast</td>
<td>0.8 mm</td>
</tr>
</tbody>
</table>

The anchor model hysteresis results are shown for the expansion anchor and undercut anchor in Figure 9.12 and Figure 9.13 respectively. The following behavioral characteristics
are observed: (1) loading along the initial stiffness branch, (2) loading along the mean monotonic backbone curve, (3) unloading along a branch that has a user defined stiffness obtained from load cycling tests. If the unloading/reloading stiffness is unknown from load cycling tests then it is recommended to set it equal to infinity for an expansion anchor and the initial stiffness for an undercut anchor, (4) reloading along the initial stiffness line, and (5) continued loading along the mean monotonic backbone curve. The steps continue in this manner including load cycling along the descending branch of the mean load-displacement backbone curve. Note that the load-displacement behavior is zoomed in on small displacements to distinguish the individual load cycles.

Figure 9.12 Hysteresis model for an expansion anchor with pull-through failure mode
9.3.4 Numerical-Experimental Comparison

The experimental results are compared with the numerical results in this section to demonstrate the ability of the model to reproduce anchor hysteretic response for different anchor types, failure mechanisms, anchor locations and different maximum crack widths. The anchor model results displayed in Figure 9.14 were developed using the $N$-$\delta$ backbone for cracked concrete for the expansion anchor. The numerical results follow the monotonic backbone curve and unload with a similar stiffness as the experimental results. The anchor model results displayed in Figure 9.15 were developed using the $N$-$\delta$ backbone for cracked concrete for the undercut anchor. The numerical results follow the monotonic backbone curve and unload with a similar stiffness as the experimental results. It is noted that the model can be modified if the monotonic test data is available for other crack widths is available. The author also recommends that an interpolation between the $N$-$\delta$ curves can be performed to model...
other crack widths. The current goal of the model is to determine how well system results can be predicted using an average monotonic backbone curve generated using a constant crack width.

Figure 9.14 Comparison of numerical and experimental load-displacement results for an expansion anchor with pull-through failure mode in cyclic cracks with \( w_{\text{max}} = 0.5 \) mm

![Comparison of numerical and experimental load-displacement results for an expansion anchor with pull-through failure mode in cyclic cracks with \( w_{\text{max}} = 0.5 \) mm](image-url)
Figure 9.15 Comparison of numerical and experimental load-displacement for an undercut anchor with breakout failure mode in cyclic cracks with $w_{\text{max}} = 0.8$ mm

9.4 Numerical Model of the Anchored System

The nonlinear element anchor model was implemented into a system model of the anchored WALLE and used to predict anchor response in terms of anchor load, $N$, and anchor displacement, $\delta$, as well as system response quantities, such as inertial system force, $F$, and displacement, $\Delta$, response of the WALLE center of mass (Figure 9.16). The system model numerical results are then compared to experimental results and a parametric sensitivity study is performed to determine the critical modeling parameters. The system model can also be extended to study other parameters that were not considered by the experimental design.
The model of the base of WALLE is shown in Figure 9.17. The model incorporates the previously developed anchor-hook model and in addition uses a gap element to model the contact between the base of the SAMU and the concrete. Since WALLE behavior was primarily observed to be a rocking type behavior of a rigid baseplate, the concrete gap element was located at the toe edge of the SAMU. If the NCS being modeled has a flexible baseplate, then a series of gap elements below the baseplate could be used to model the contact with concrete. In SAP2000 the nonlinear gap element allows the user to input a spring stiffness constant, therefore the gap element/concrete compression spring can be handled by one nonlinear gap element.
Figure 9.18 summarizes the user defined properties for nonlinear gap and hook elements in SAP2000 (units: kips, inches). The hook element, named LSP1 (link support) is the same as described in the previous section. The purpose of the gap element, named Comp, is to model the compression only contact between the baseplate and the concrete. In the case of the WALLE it is the contact between the toe of the SAMU and the concrete. The WALLE SAMUs and basetube are very stiff and are therefore modeled as a rigid baseplate with concrete gap elements only at the “toe” of the SAMUs which were observed during the experiments to be the points of rotation/rocking of the WALLE.
The SAP2000 model of the anchored WALLE is shown in Figure 9.19. Joints were located where masses and response quantities are desired. For example at the anchors, SAMU toe, Accelerometers: Accl3, Accl4, Accl5, the center of mass as well as the center of the mass plates. All joints were restrained in the transverse (Y-direction). The four anchors were restrained in the X-direction to resist the base shear, ignoring shear load transfer between the WALLE base and the concrete. The self weight of the HSS sections was internally calculated by SAP2000 and incorporated into the mass matrix. A translational mass of 0.0621 kip-s²/ft (Weight = 2 kips) was lumped at the centerline of the weight plates. Because the size of the weight plates was large, a rotational mass moment of inertia of 0.024 kip-ft-s² (Weight moment of inertia = 0.781 kip-ft²) was assigned about the Y-axis at the centerline of weight plates.
The WALLE was modeled as linear elastic with all nonlinear action concentrated at the anchor elements and nonlinear gap elements. Elastic beam elements labeled “MAST” were used for the mast that had the properties of a 4”x4”x3/8” HSS. Elastic beam elements labeled “BOX” were used for the transverse tube that had the properties of a 6”x6”x1/2” HSS. The SAMUs and the bottom portion of the mast were assigned “RIGID” element properties in order to model the effect of the stiffener plate at the base of the mast as well as a rigid baseplate (Figure 9.20).
Based on findings from the experimental results and recommendations from literature (ASCE 43, 2005) the base case numerical model uses the following input parameters. These parameters are varied later in a sensitivity study to examine their effect on the model results.

- Elastic model of WALLE
- Anchor monotonic load-displacement backbone obtained from testing in cracked concrete
- Viscous damping of 2% of critical
- “Normal” concrete gap element stiffness of 500 kip/in
9.5 Numerical Model Results

The numerical simulation results from the SAP2000 system model are summarized in the following sections. Simulation results are compared with experimental results for tests conducted on the expansion anchor and undercut anchor (Table 9.2). The tests include incrementally increasing seismic input where the anchor response ranges from essentially elastic to moderate inelastic response and failure tests. The results are grouped in the following categories: (i) displaced shape, (ii) time history results, and (iii) maximum response parameter results. The results use an anchored WALLE model that consider, 2% viscous damping, cracked concrete monotonic load-displacement backbone, and concrete contact stiffness modeled using a compression stiffness of 500 kip/in. The parameter set was selected as an initial starting point without model calibration. Two percent viscous damping was selected based on the system identification results. Use of a cracked monotonic envelope was based on the results and recommendations of the WALLE dynamic test results. These parameters are later systematically varied in a parametric sensitivity study to determine the sensitivity of the model results to their input. However, before the results are presented, the target accuracy of the numerical model should be discussed.
Table 9.2 SAP2000 baseline simulation test list

<table>
<thead>
<tr>
<th>Anchor Type (Crack width)</th>
<th>Simulation No.</th>
<th>Simulation Name</th>
<th>Experiment Test Name</th>
</tr>
</thead>
<tbody>
<tr>
<td>Expansion Anchor (w = 0.5 \text{ mm})</td>
<td>1</td>
<td>V2.WALLE.F.N5.CC05.10P</td>
<td>V2.WALLE.F.N5.CC05.FM02.10P</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>V2.WALLE.F.N5.CC05.30P</td>
<td>V2.WALLE.F.N5.CC05.FM02.30P</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>V2.WALLE.F.N5.CC05.100P</td>
<td>V2.WALLE.F.N5.CC05.FM02.100P</td>
</tr>
<tr>
<td>Expansion Anchor (w = 0.8 \text{ mm})</td>
<td>4</td>
<td>WALLE.F.N5.CC08.30P</td>
<td>WALLE.F.N5.CC08.FM02.30P</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>WALLE.F.N5.CC08.100P</td>
<td>WALLE.F.N5.CC08.FM02.100P</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>WALLE.F.N5.CC08.125P</td>
<td>WALLE.F.N5.CC08.FM02.125P</td>
</tr>
<tr>
<td>Undercut Anchor (w = 0.8 \text{ mm})</td>
<td>7</td>
<td>WALLE.F.N9.CC08.20P</td>
<td>WALLE.F.N9.CC08.FM02.20P</td>
</tr>
<tr>
<td></td>
<td>8</td>
<td>WALLE.F.N9.CC08.40P</td>
<td>WALLE.F.N9.CC08.FM02.40P</td>
</tr>
<tr>
<td></td>
<td>9</td>
<td>WALLE.F.N9.CC08.60P</td>
<td>WALLE.F.N9.CC08.FM02.60P</td>
</tr>
<tr>
<td></td>
<td>10</td>
<td>V2.WALLE.F.N9.CC08.20P</td>
<td>V2.WALLE.F.N9.CC08.FM02.20P</td>
</tr>
<tr>
<td></td>
<td>11</td>
<td>V2.WALLE.F.N9.CC08.40P</td>
<td>V2.WALLE.F.N9.CC08.FM02.40P</td>
</tr>
<tr>
<td></td>
<td>12</td>
<td>V2.WALLE.F.N9.CC08.60P</td>
<td>V2.WALLE.F.N9.CC08.FM02.60P</td>
</tr>
</tbody>
</table>

9.5.1 Model Target Accuracy

The accuracy of the model results must be viewed in light of the aleatory variability of the physical experimental results. There are several sources of variability in the achieved anchor loads and displacements as follows:

- Variability in applied anchor loading due to uneven load sharing between a pair of anchors on one side of WALLE (\(CV_s: 20-30\%\))
- Variability in anchor ultimate capacity due to strain rate effects (\(CV_R: 5-10\%\))
- Variability in anchor mean ultimate load, \(N_{u,m}\) (\(CV_N: 5-20\%\))
- Variability in anchor mean displacement at ultimate load, \(\delta_{u,m}\) (\(CV_d: 20-50\%\))

The individual variability’s may be estimated estimated and then combined using a propagation of uncertainty approach to obtain an estimate of the composite variability on anchor load and anchor displacement (e.g. NIST, 2011). The experimental composite variability provides a point of reference for evaluating the scatter in the numerical model results.
Variability due to load sharing, $CV_S$

Based on a study of load sharing on a pair of anchor on one side of WALLE it was found that the load on a pair of anchors is not evenly shared 50/50. At low levels of load the variability is large and tends to decrease with larger anchor load. Considering $N_{\text{max},k} > 5\% N_{um,cr}$ the variability was approximately 40% and considering $N_{\text{max},k} > 25\% N_{um,cr}$ the variability was approximately 30%. Because anchor failure occurs at large loads, the $CV_S$ due to load sharing was estimated to be 30%.

Variability due to strain rate, $CV_R$

Hoehler et al. (2011a) discuss the behavior of anchors in concrete at seismic-relevant loading rates. They conclude that anchor ultimate strength for concrete breakout failure will be at least as large as that under quasi-static loading and on average will be 30% larger. They conclude that for pull-through failure, earthquake relevant load rates do not decrease the ultimate strength, but does not show a consistent trend above the normal scatter of the results. The plot of the data however has a mean strength greater than the static strength and is estimated to be approximately $1.2 N_{u,m(\text{static})}$. However, since a detailed study on variability of anchor strength due to strain rates has not been published it is difficult to assess. In the author’s opinion the variability due to strain rates may be estimated to be 5% as a lower bound minimum estimate.

Variability in ultimate load, $CV_N$

From the monotonic test results, the coefficient of variation, $CV_N$, on anchor mean ultimate tension load, $N_{u,n}$, ranges from 5-20% and is on average 12%.
Variability in ultimate displacement, CV\(_{\delta}\)

From the monotonic test results, the coefficient of variation, CV\(_{\delta}\), on mean anchor displacement at ultimate load, \(\delta_{u,m}\), ranges from 20-50% and is on average 30%.

Composite variability in anchor load, CV\(_{N,\text{composite}}\)

Adopting a propagation of uncertainty approach to obtain an estimate of the composite variability on anchor load (e.g. NIST, 2011), the individual variabilities may be combined using a square root sum of squares combination as follows. This leads to a composite variability on anchor load of 33%.

\[
CV_{N,\text{composite}} = \sqrt{CV_{S}^2 + CV_{R}^2 + CV_{\delta}^2}
\]

\[
CV_{N,\text{composite}} = \sqrt{0.3^2 + 0.05^2 + 0.12^2} = 0.33
\]

Composite variability in anchor displacement, CV\(_{d,\text{composite}}\)

Adopting a propagation of uncertainty approach to obtain an estimate of the composite variability on anchor displacement (e.g. NIST, 2011), the individual variabilities may be combined using a square root sum of squares combination as follows. This leads to a composite variability on anchor displacement of 43%.

\[
CV_{d,\text{composite}} = \sqrt{CV_{S}^2 + CV_{R}^2 + CV_{N}^2}
\]

\[
CV_{d,\text{composite}} = \sqrt{0.3^2 + 0.05^2 + 0.3^2} = 0.43
\]
**Composite variability in WALLE response**

For an anchored WALLE, where the WALLE and the SAMU base plates remain elastic, with the primary non-linearity coming for the inelastic actions in the anchors, it may be inferred that the WALLE system force and displacement will have a variability greater than or equal to the variability on the anchor response quantities. The WALLE displacement variability should be commensurate with the variability of anchor displacement and the WALLE acceleration variability should be commensurate with the variability of anchor force.

**9.5.2 Displaced shape**

To verify that a dynamic model is working properly it is useful to examine the modal properties and displaced shape. Figure 9.21 summarizes the first and second mode shapes and modal frequencies for the WALLE anchored in uncracked concrete with expansion anchors. The first and second modal frequencies are 4.5 Hz and 37 Hz respectively. This is comparable to the experimentally obtained modal frequencies from white noise tests of 4.6 Hz and 35 Hz for the first and second modes respectively.
The elastic WALLE displacements are a small component of the total WALLE displacement. The nonlinear displacement behavior of the anchors highly contribute to the overall displacement of the WALLE. Several stages of anchor displacement, and thus WALLE displacement, may be identified. The stages of anchor displacement are as follows: (i) the anchors respond elastically at low levels of seismic input (Figure 9.22), (ii) the anchors begin to experience non recoverable inelastic displacements (Figure 9.23), (iii) the WALLE reverses direction and causes inelastic displacements of the pair of anchors on the opposite side of WALLE (Figure 9.24), this process continues as the WALLE rocks back and forth with increasing incremental anchor displacement during strong input motion, then (iv) the WALLE response dies out and the anchors are left in a permanently displaced position, having residual displacements (Figure 9.30).
Figure 9.22 Displaced shape of WALLE base showing elastic deformation of anchors

Figure 9.23 Displaced shape of WALLE base showing inelastic deformation of anchors
Figure 9.24 Displaced shape of WALLE base showing anchor pull-through displacements

Figure 9.25 Displaced shape of WALLE base after test showing residual displacement of anchors

9.5.3 Nonlinear Time History Analysis Results

The results of the numerical model can be compared with the experimental results to make meaningful evaluations of the predictive capability of the model. The following anchor
and system response parameters are of interest: (i) WALLE acceleration at the center of mass, (ii) WALLE displacement at the center of mass, (iii) anchor tension load, $N$, (iv) anchor axial displacement, $\delta$, and (v) anchor load displacement response, $N-\delta$. In what follows, the time histories, as well as maximum quantities of these parameters are evaluated. Results are shown for low to high seismic demand cases including the failure tests for the expansion and undercut anchors. The purpose is to demonstrate various stages of anchor response from primarily elastic to moderately inelastic, to highly inelastic at failure. Table 9.3 lists the tests that are presented herein to demonstrate the baseline model capabilities.

<table>
<thead>
<tr>
<th>Anchor / Crack width</th>
<th>Test Name &amp; (Figure No.)</th>
<th>Seismic Input Level (Scale)</th>
<th>Anchor Behavior</th>
</tr>
</thead>
<tbody>
<tr>
<td>Expansion ($w = 0.5$ mm)</td>
<td>V2.WALLE.F.N5.CC05.30P (Figure 9.26)</td>
<td>Moderate 30%</td>
<td>moderately inelastic behavior, mild pull-through, $\delta_{\text{max}} = 0.15&quot;$ (4 mm)</td>
</tr>
<tr>
<td></td>
<td>V2.WALLE.F.N5.CC05.100P (Figure 9.27)</td>
<td>Maximum failure test 100%</td>
<td>highly inelastic behavior at anchor failure, pull-through failure occurred, $\delta_{\text{max}} = 1.5&quot;$ (38 mm)</td>
</tr>
<tr>
<td>Undercut ($w = 0.8$ mm)</td>
<td>WALLE.F.N9.CC08.20P (Figure 9.28)</td>
<td>Low 20%</td>
<td>mostly elastic behavior, $\delta_{\text{max}} = 0.04&quot;$ (1 mm)</td>
</tr>
<tr>
<td></td>
<td>WALLE.F.N9.CC08.40P (Figure 9.29)</td>
<td>Moderate 30%</td>
<td>mild inelastic behavior, $\delta_{\text{max}} = 0.06&quot;$ (1.5 mm)</td>
</tr>
<tr>
<td></td>
<td>WALLE.F.N9.CC08.60P (Figure 9.30)</td>
<td>Maximum Failure test 60%</td>
<td>highly nonlinear inelastic behavior at anchor failure, concrete breakout occurred $\delta_{\text{max}} = 0.3&quot;-1&quot;$ (8-25 mm)</td>
</tr>
</tbody>
</table>

Figure 9.26 and Figure 9.27 compare the results of the numerical model with the experimental results for selected seismic tests of the expansion anchor in cycled cracks having a maximum width of 0.5mm (0.02”). Similarly, Figure 9.28 and Figure 9.30 compare the results of the numerical model with the experimental results for selected seismic tests of the undercut anchor in cycled cracks having a maximum width of 0.8mm (0.03”). It is noted that
the anchor load and anchor displacement of the South pair of anchors is plotted negative for clarity. The input motion scale ranges from 20% to 100% so that the model results can be compared for cases where the anchorage remain relatively elastic (small displacements) as well as when significant inelasticity was experienced by the anchors, including failure. The last test of each series is the failure test where there is significant period elongation that occurs in the system due to anchor displacement. It is noted that the expansion anchor failed by pull-through failure while the undercut anchor failed by concrete breakout.

The plots compare the system response parameters; namely, WALLE acceleration and WALLE displacement at the center of mass (CM) as well as anchor response parameters, anchor axial tension load, \( N \), anchor axial displacement, \( \delta \), versus time and the load-displacement, \( N-\delta \), behavior of the individual anchors. The maximum response values are identified and listed in the plots. It is observed that the numerical model time history results generally track the experimental results. In some cases the maximum values occur on a different load-cycle; however, the maximum values are comparable. In design practice, it is the maximum value that is typically the governing case. As is typical with nonlinear analysis, small changes in initial conditions can cause drastic changes in outcome. This can be observed in Figure 9.30, the failure test of the undercut anchor. Due to the asymmetric nature of the input motion, the experimental results predict the maximum WALLE displacement to the South, thus causing maximum displacement in the North pair of anchors, while the experimental results, WALLE maximum displacement occurs to the North. However, when the absolute value of the maximum values are compared, the maximum response parameters that would be used in design are generally comparable. For example the maximum experimental anchor displacement is approximately 1.05” on the South anchor; whereas the maximum simulated anchor displacement is approximately 0.90” on the North anchors. It is
noted that these displacements are significantly larger, by approximately 800%, than the mean ultimate monotonic displacement, $\delta_{um,cr}$, of 0.111", therefore the numerical model adequately predicts maximum response quantities well into the inelastic range and past the point of anchorage failure. This is a significant finding for a relatively simple nonlinear model of tension dominated anchor behavior for load cycling dominant applications. It is noted that in the following plots, anchor load and anchor displacement of the South pair of anchors is plotted negative for clarity.
Figure 9.26 Comparison of numerical and experimental results, expansion anchor, $w_{max}=0.5\text{mm}, 30\%$ input scale, V2.WALLE.F.N5.CC05.30P
Figure 9.27 Comparison of numerical and experimental results, expansion anchor, $w_{max}=0.5\text{mm}$, 100% input scale, V2.WALLE.F.N5.CC05.100P
Figure 9.28 Comparison of numerical and experimental results, undercut anchor, \( w_{\text{max}} = 0.8 \text{mm}, 20\% \text{ input scale}, \text{WALLE.F.N9.CC08.20P} \)
Figure 9.29 Comparison of numerical and experimental results, undercut anchor, $w_{\text{max}}=0.8\text{mm}$, 40% input scale, WALLE.F.N9.CC08.40P
Figure 9.30 Comparison of numerical and experimental results, undercut anchor, \( w_{\text{max}}=0.8 \text{mm}, 60\% \text{ input scale}, \text{WALLE.F.N9.CC08.60P} \)
9.5.4 Maximum Response Results

Seismic design practice for anchorage and nonstructural components is typically focused on maximum response quantities. In this case, maximum load and displacement of the anchors and maximum acceleration and displacement of the NCSs. Therefore this section focuses on comparing maximum predicted response from the numerical simulations with those measured during the experiments. Figure 9.31 compares the maximum experimental anchor load, $N_{\text{max}}$, with the maximum anchor load from analysis for the tests identified in Table 9.2 for the expansion and undercut anchor. The maximum anchor load in the simulation case saturates at the mean maximum value from the monotonic backbone, $N_{\text{um,cr}}$, whereas the experimental anchor load can increase to the mean uncracked maximum load, $N_{\text{um,uncr}}$, when the crack is smaller or due to strain rate effects. This effect can be more clearly seen when the load results are normalized by $N_{\text{um,uncr}}$ (Figure 9.32). From the cases shown, it is noted that in general the maximum anchor load is underpredicted in the numerical simulations. This is because the anchor ultimate load in cracked concrete is a lower bound on anchor capacity.
The maximum anchor displacement, $\delta_{\text{max}}$, from simulations is compared with the experimental results (Figure 9.33). Recalling that the mean anchor displacement from
monotonic tests, $\delta_{um,cr}$ was approximately 0.50” (13 mm) for the expansion anchor in both crack widths 0.5mm and 0.8mm, and was approximately 0.11” (3 mm) for the undercut anchors, the simulation model predicts anchor displacements reasonably in the low displacement range and for very large anchor displacements well past failure for the test cases considered. This displays the ability of the anchored-WALLE simulation model to predict WALLE rocking behavior after large anchor displacements have occurred. This can be seen more clearly in Figure 9.34 where anchor displacements were predicted well up to 5 times $\delta_{um,cr}$ and reasonably up to 10 times $\delta_{um,cr}$, but with increased scatter.

Figure 9.33 Maximum anchor displacement, simulation vs. experiment
Considering that the anchor displacements are well predicted by the model it can also be expected that the system displacements would also be well predicted by the SAP2000 model. Figure 9.35 compares the maximum numerical and experimental WALLE displacement at the center of mass in the direction of shaking. The maximum WALLE displacements are predicted well even at large displacements (7” to 11”). Considering that the WALLE center of mass, CM, is located at a height of 55”, the drift ratio (Figure 9.36) can be calculated as the displacement of the CM over the height to the CM. The numerical model predicts drift ratio reasonably well even at large drift ratios (12% to 21%). It is also noted that the calculated drift ratios are sensitive to the model NCS geometry used in these experiments and should not be generalized to all NCSs.
Figure 9.35 Maximum displacement at WALLE center of mass simulation vs. experiment

Figure 9.36 Maximum drift ratio at WALLE center of mass simulation vs. experiment

Figure 9.37 compares the experimental and numerical maximum WALLE acceleration at the center of mass. The numerical model generally over predicted the experimental
accelerations. The amount of over prediction increased with increasing acceleration which suggests that there was a saturation point for experimental accelerations that is not represented in the numerical model. It is also noted that maximum accelerations tend to be caused by high frequency spikes in the acceleration signal which do not strongly influence overall dynamic response. This may be one reason why the numerical model is able to closely predict other response quantities even though maximum acceleration tends to be over predicted.

![Graph showing maximum acceleration at WALLE center of mass simulation vs. experimental](image)

**Figure 9.37 Maximum acceleration at WALLE center of mass simulation vs. experimental**

### 9.5.5 Parametric Sensitivity Study

In addition to the model geometry and elastic material properties, there are three important parameters that must be selected as input to the SAP2000 model, which are as follows: (i) viscous (Rayleigh) proportional damping, defined as constant for all modal frequencies, (ii) anchor monotonic backbone curve, cracked or uncracked and (iii) stiffness of
the concrete gap element. These parameters were studied to determine the sensitivity of the model response to their selection. The other modeling decisions such as elastic stiffness of WALLE, mass distribution of WALLE as well as model mesh density are consistent with commonly accepted approaches and are therefore not discussed herein. Table 9.4 provides an overview of the combinations of parameters that were varied during the parametric sensitivity study and the purpose of each parameter set.

Table 9.4 Sensitivity study parametric combinations

<table>
<thead>
<tr>
<th>Set #</th>
<th>Viscous Damping</th>
<th>Cracked or Uncracked Backbone</th>
<th>Concrete Gap Element Stiffness</th>
<th>Purpose</th>
<th>No. of Simulation runs</th>
</tr>
</thead>
<tbody>
<tr>
<td>#1 base-line</td>
<td>2%</td>
<td>Cracked w=0.8mm</td>
<td>Normal 500 kip/in</td>
<td>Determine if the cracked or uncracked monotonic mean backbone better predicts anchor and NCS response parameters</td>
<td>12</td>
</tr>
<tr>
<td>#2</td>
<td>2%</td>
<td>Uncracked w=0.0mm</td>
<td>Normal 500 kip/in</td>
<td>Determine if the cracked or uncracked monotonic mean backbone better predicts anchor and NCS response parameters</td>
<td>12</td>
</tr>
<tr>
<td>#3</td>
<td>2%</td>
<td>Cracked</td>
<td>Stiff 10,000 kip/in</td>
<td>Determine the influence of concrete gap element stiffness on anchor and NCS response parameters</td>
<td>12</td>
</tr>
<tr>
<td>#4</td>
<td>5%</td>
<td>Cracked</td>
<td>Normal 500 kip/in</td>
<td>Determine the influence of equivalent viscous damping on anchor and NCS response parameters. Note: hysteretic damping due to anchor nonlinear behavior is explicitly modeled. The viscous damping covers other sources of damping.</td>
<td>12</td>
</tr>
</tbody>
</table>

Total: 48
Table 9.5 Parametric study: list of numerical simulation runs

<table>
<thead>
<tr>
<th>Anchor Type (Crack width)</th>
<th>Simulation No. &amp; Parameter Set</th>
<th>Simulation Name</th>
<th>Experimental Test Name</th>
</tr>
</thead>
<tbody>
<tr>
<td>Expansion Anchor (w = 0.5 mm)</td>
<td>set#1</td>
<td>V2.WALLE.F.N5.CC05.10P</td>
<td>V2.WALLE.F.N5.CC05.10P</td>
</tr>
<tr>
<td></td>
<td>set#2</td>
<td>V2.WALLE.F.N5.CC05.10P_uc</td>
<td>V2.WALLE.F.N5.CC05.10P</td>
</tr>
<tr>
<td></td>
<td>set#3</td>
<td>V2.WALLE.F.N5.CC05.10P_hard</td>
<td>V2.WALLE.F.N5.CC05.10P</td>
</tr>
<tr>
<td></td>
<td>set#4</td>
<td>V2.WALLE.F.N5.CC05.10P_d5</td>
<td>V2.WALLE.F.N5.CC05.10P</td>
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<tr>
<td></td>
<td>set#5</td>
<td>V2.WALLE.F.N5.CC05.30P</td>
<td>V2.WALLE.F.N5.CC05.30P</td>
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<tr>
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<td>V2.WALLE.F.N5.CC05.30P_hard</td>
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<tr>
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<td>set#8</td>
<td>V2.WALLE.F.N5.CC05.30P_d5</td>
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<td>set#9</td>
<td>V2.WALLE.F.N5.CC05.100P</td>
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<td>V2.WALLE.F.N5.CC05.100P</td>
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<tr>
<td>Expansion Anchor (w = 0.8 mm)</td>
<td>set#13</td>
<td>WALLE.F.N5.CC08.30P</td>
<td>WALLE.F.N5.CC08.30P</td>
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<td>set#14</td>
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<td>set#15</td>
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<td>WALLE.F.N5.CC08.30P</td>
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</tr>
</tbody>
</table>

Parameter Set #1: 2% damping, cracked concrete backbone, normal gap stiffness
Parameter Set #2: 2% damping, uncracked concrete backbone, normal gap stiffness
Parameter Set #3: 2% damping, cracked concrete backbone, stiff "hard" gap stiffness
Parameter Set #4: 5% damping, cracked concrete backbone, normal gap stiffness
Figure 9.38 summarizes the maximum anchor load during the parametric sensitivity study. The major difference from the cracked and uncracked concrete monotonic backbone curves is the ability of the uncracked concrete case to better predict large anchor loads in excess of $N_{\text{um,cr}}$. These loads typically occurred when the cracks were closed. The model using 5% damping almost always underpredicts the response. The stiff gap model had negligible effect on anchor forces while the uncracked model typically overpredicted the maximum anchor load.

![Figure 9.38 Maximum anchor load parametric study](image)

Figure 9.39 summarizes the maximum anchor displacement during the parametric sensitivity study. It can be concluded that the baseline (original) parameter set #1 (2% damping, cracked monotonic envelope, and normal gap stiffness) most closely predicted maximum anchor displacement. It is observed that using the uncracked concrete monotonic
envelope in some cases overestimated and in some cases underestimated the maximum anchor displacements. The overestimations are cases when larger forces developed in the anchors causing more force to be transmitted to WALLE so that when the anchors did begin to pass their failure displacement the total energy of the WALLE mass was larger leading to increased maximum anchor displacements. The underestimations occurred during lower input motion tests where because of the larger maximum load capacity, the anchors did not displace as much. The higher level of damping, 5% versus 2%, caused the model to generally under predict maximum anchor displacement during the low level tests and had less of an effect at the higher input scale tests where WALLE rocking behavior dominated the response.

Figure 9.39 Maximum anchor displacement parametric study

Figure 9.40 summarizes the maximum WALLE drift ratio, determined at the WALLE center of mass, during the parametric sensitivity study. Again, the model using parameter set #1 (2% damping, cracked monotonic envelope, and normal gap stiffness) most closely
predicted maximum drift ratio. Parameter set #4, 5% damping, tended to under predict WALLE drift ratio in all tests. Parameter set #3, stiff concrete gap spring, did not significantly change the results from parameter set #1, normal gap stiffness, demonstrating that anchor displacements are not very sensitive for the two concrete gap spring stiffness’s studied.

![Graph](image)

**Figure 9.40 Maximum drift ratio at WALLE center of mass parametric study**

Figure 9.41 summarizes the maximum WALLE acceleration, determined at the WALLE center of mass, during the parametric sensitivity study. All parameter sets generally overestimate the WALLE acceleration response for all tests. Parameter set #2, uncracked backbone and parameter set #3, stiff gap tended to more significantly over predict WALLE maximum acceleration response. This is because the anchors modeled using the uncracked backbone tend to attract higher forces which then transmit these forces up into the WALLE. Similarly the stiff concrete spring causes too hard of an impact when the SAMUs contact the concrete thereby transmitting larger accelerations up into WALLE. Based on these results, it is recommended to use parameter set #1.
9.6 Parameter Selection Recommendations

Based on the results of the sensitivity study, it is recommended to use the anchor model input parameter ranges listed in Table 9.6 and the NCS system input parameter ranges listed in Table 9.7. The analyst is cautioned to carefully consider the physical conditions of the anchored system being modeled and have a clear understanding of the expected behavior of the system before extending these modeling procedures to situations beyond what is covered in this study.
Table 9.6 Anchor model parameter selection

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Selection Criteria &amp; Range</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Anchor Model Parameters</strong></td>
<td></td>
</tr>
<tr>
<td><strong>Parameter</strong></td>
<td><strong>(Recommended Value)</strong></td>
</tr>
<tr>
<td>Load-deformation curve</td>
<td>(mean monotonic tension envelope from cracked concrete with constant crack width ( w = w_{\text{max,cyclic}} ))</td>
</tr>
<tr>
<td>Hook element stiffness</td>
<td>(1000 kip/in)</td>
</tr>
<tr>
<td>Hook element initial gap</td>
<td>(open = 0)</td>
</tr>
<tr>
<td>Pivot parameter, ( \alpha )</td>
<td>(( \alpha = 3000 ))</td>
</tr>
<tr>
<td>Gap element stiffness</td>
<td>(500 kip/in)</td>
</tr>
<tr>
<td>Gap element initial gap</td>
<td>(closed = 0)</td>
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### Table 9.7 NCS model parameter selection

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Selection Criteria &amp; Range</th>
</tr>
</thead>
<tbody>
<tr>
<td>Elastic / Inelastic NCS (Elastic WALLE)</td>
<td>Use an NCS component model that is appropriate for the component and stress level Use a damping level appropriate for the NCS construction and strain level. Recommendations for various NCS types and response levels are given by Table 3-2 of ASCE 43, 2005. For welded metal structures and massive low stressed mechanical components at response level 1 (Demand/Capacity = D/C ≤ 0.5) use 2%. The sensitivity study herein considered 2 and 5% damping and found that 2% better matches experimental results</td>
</tr>
<tr>
<td>Damping (2%)</td>
<td>Consider using if large deformations are expected where P-Delta effects could significantly affect behavior Use a concrete gap spring stiffness that is at least 10 times stiffer than the baseplate, but not stiffer than the connected concrete element (EA/L) stiffness. Sensitivity study used 500 and 10000 kip/in and found 500 kip/in better matched experimental results</td>
</tr>
<tr>
<td>P-Delta (On)</td>
<td>Use a concrete gap spring stiffness that is at least 10 times stiffer than the baseplate, but not stiffer than the connected concrete element (EA/L) stiffness. Sensitivity study used 500 and 10000 kip/in and found 500 kip/in better matched experimental results</td>
</tr>
<tr>
<td>Concrete gap stiffness (500 kip/in)</td>
<td>The initial gap can be used to model NCS feet are not in full contact with the concrete initially. In the WALLE tests, the SAMUs were carefully leveled and shimmed to create a contact surface with the concrete. Therefore the initial gap was set to zero (e.g. tight contact between the base plate and concrete))</td>
</tr>
<tr>
<td>Gap element initial gap (open = 0)</td>
<td>Very stiff “rigid” links were used to model the SAMUs as they were constructed of 6”x6”x1” angle with stiffener plates. For applications in practice, baseplate flexibility may be important, in which case multiple gap springs could be used to model the contact condition with concrete along the length of the plate</td>
</tr>
<tr>
<td>Rigid / Flexible Baseplate (Rigid)</td>
<td>It is up to the modeler to input all masses of the NCS as well as any additional masses. Consideration should be given to rotational mass moment of inertial if it is deemed to be significant</td>
</tr>
<tr>
<td>Mass (translation &amp; rotation)</td>
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</table>
load cycling dominant applications. The anchor model was then extended for use in a system model of the anchored-WALLE system to predict NCS force-deflection responses. The nonlinear anchor model has the following characteristics: (i) the model is calibrated against single anchor test data (ii) the model can be implemented in commonly available nonlinear analysis software packages, in this case SAP2000, and is therefore practical to use in a design office setting where nonlinear analysis is routinely performed, (iii) the model extends the application of the pivot hysteresis rule developed by Dowell et al (1997) for reinforced concrete member behavior to anchor modeling and (iv) the model yields reasonable results that can be used for anchorage and NCS design without the need for a 3D FEA phenomenological model of anchor mechanisms, such as bearing, friction, slip, concrete cracking, etc,…

The numerical results demonstrate that a relatively simple model of anchor behavior based on a nonlinear element which uses the pivot hysteresis rule, in series with a tension-only “hook” element, may be used to adequately model complex anchor nonlinear behavior. The results also demonstrate that the local anchor model can be implemented into an anchor-NCS system model to obtain reasonable estimates of peak response quantities of anchor load, \( N \), anchor displacement, \( \delta \), system force \( F \), and system displacement, \( \Delta \), that occur during seismic inertial loading of an NCS anchored in crack cycling boundary conditions.

**9.7.1 Limitations of the Model**

The numerical model was developed for and compared against the experimental results from the WALLE dynamic tests and therefore has the following limitations:

- The model does not account for shear load-deformation of the anchor, therefore it is limited to *tension dominated* systems with large anchor tension to shear ratios. Adapting the ACI 318 code recommendations, a tension
A dominated system is one in which the shear force demand is less than 20% of the shear capacity of the anchor.

- The model is limited to load cycling dominated cases rather than crack cycling dominated cases. A typical load cycling dominated case is a floor mounted NCS that has a natural frequency, and thus anchor loading frequency, that is larger than the frequency of cracking where the anchor is attached to the concrete substrate (e.g. slab or beam in the structure). This is the common case for typical floor mounted, non-isolated NCSs in typical buildings, as they have a high natural frequency compared with the predominant natural frequency of a typical building.

- The model requires as input the mean monotonic load-displacement curve from anchor testing. Fortunately, due to product qualification testing requirements for use of anchors in cracked concrete in seismic applications this data can be obtained from manufacturers or single anchor tension tests.

- The model was evaluated using tests conducted with expansion and undercut anchors having pull-through and concrete breakout failures respectively. However, based on the study results the anchor and system behavior are dependent on the shape of the load-deflection curve rather than the physical anchor mechanism that caused the curve. In that respect, the model may be applied to other anchor types/failure mechanisms where the load-displacement behavior is similar to the anchors studied.

- The model was developed and compared to test results where the NCS and baseplate remain elastic and all nonlinear behavior is confined to the anchors.
Careful consideration should be given if the NCS is expected to experience nonlinear behavior in the NCS itself.

### 9.7.2 Summary of Model Results

Based on the numerical model results, the following conclusions can be made:

- Anchor nonlinear behavior in tension load cycling dominated situations can be adequately represented using a simple *line* element that is a combination of a tension-only link element and a uniaxial nonlinear element. In this case, the pivot model (Dowell et al., 1998) is extended to model nonlinear anchor hysteresis behavior in tension load cycling dominated situations. The nonlinear anchor element can be used to adequately predict anchor load, $N$, and anchor displacement, $\delta$.

- The anchor model takes a complex physical mechanism on the *micro* level that involves concrete bearing, local concrete crushing, concrete cracking, friction between steel-to-steel elements, and friction between steel-to-concrete elements and simplifies the problem to the *macro* level of anchor load-displacement behavior ($N-\delta$).

- For anchors located in cyclic cracked concrete boundary conditions the mean monotonic envelope from tension testing should be used as input to the anchor pivot model. The anchor unloading stiffness should be determined from load cycling tests. If load cycling tests are not available, it is recommended to use a very large (almost vertical) unloading stiffness for expansion anchors and an unloading stiffness equal to the initial stiffness of the anchor for undercut anchors.
• An advantage of the anchor model is that the input to the anchor hysteresis model is obtained from commonly performed monotonic tension tests, therefore the modeling procedure is applicable to a wide range of anchors.

• The model was developed and validated using results from expansion and undercut anchors having pull-through and concrete breakout failures respectively; however, the model can be extended to other anchor types with similar load-displacement characteristics.

• The nonlinear anchor model can also be incorporated into a model of an anchored NCS system to adequately predict maximum system response in terms of NCS displacement, $\Delta$, and acceleration, $\dot{A}$. These are the typical response parameters needed for NCS design.

9.8 Future Work

This work has demonstrated that a relatively simple nonlinear anchor model can be developed and implemented into an anchored NCS system model to adequately capture the complex nonlinear seismic behavior of an anchored-NCS system. In this system the anchors experience cyclic inertial tension loads while the anchors are located in cyclic cracked concrete boundary conditions. The research also has revealed new questions to be answered. Recommendations for future work are as follows:

• Improve the anchor nonlinear model to update the anchor tension load-displacement hysteresis curves based on the instantaneous crack width at a given time.

• The model could be used to develop component response modification factors $R_p$ for anchorage systems. This could be for elastically responding
components where the energy absorption is intended to take place in the anchorage or for combined systems where inelastic action is expected in both the component and the anchorage.

- Currently, seismic qualification of nonstructural components per ICC-ES AC156 (2010) or other test standards typically considers a component that is “hard-mounted” to the shake table, either via bolting or welding. Similar to seismic isolation-restraint devices, anchorage flexibility and inelastic actions will modify the component response and hence its demands. The anchorage model could be implemented into a system model of the anchored nonstructural component in order to determine the modified component demands in terms of response accelerations and displacements.

- Based on the results of dynamic tests of shear dominated components and the test results for anchors in shear load cycling, the anchor model could be updated to include a non-linear element in the horizontal direction that would account for shear deformations in the inelastic range. This could extend the model to be applicable to NCSs that are in-between shear dominated and tension dominated.

- WALLE experimental results are also available for epoxy and drop-in anchors. It is recommended that the modeling procedure be applied to these anchors as a confirmation that the monotonic mean displacement curve can be used to adequately represent other anchor types.
Chapter 10 Impacts on Design and Future Research

The aim of this research was to investigate the seismic behavior of anchored Nonstructural Components and Systems (NCSs). During earthquake shaking, structural members in a building will suffer cracking that oscillates as the building deforms. Equipment that services the building, such as mechanical and electrical items, are anchored to these components, and therefore will be subjected to this dynamic environment. Despite understanding this practical loading situation, as well as recognizing the significant reduction in anchor load capacity due to its embedment in cracked concrete, there remains a gap in knowledge regarding the effect of anchorage behavior on nonstructural component response. In particular, the effect of dynamic cyclic cracking on the anchor and component response has not been studied to date.

This dissertation presents the results of system-level shake table tests of floor mounted nonstructural components anchored in cyclically cracked concrete subjected to seismic inertial loading. The experiments focused on three areas of investigation:

- **System identification tests** to characterize the dynamic properties of model nonstructural components and systems (NCSs) in terms of natural period and damping in both uncracked and cyclically cracked concrete.

- **Correlation tests** to determine the relationship between amplitude of anchor loading and corresponding crack width. The correlation tests were further divided into “variable-phase” correlation and “in-phase” correlation tests.

- **Failure tests** performed on four anchor types in cyclically cracked concrete. Anchor types included epoxy, torque controlled expansion, drop-in, and undercut post installed anchors.
This dissertation also presents the development of a nonlinear, lumped hysteresis macro-model of anchor behavior for predicting anchor load-displacement response for tension load cycling dominant applications. The anchor model is calibrated against single anchor tests and subsequently extended for use in a system model of the anchored NCS for predicting maximum system response.

Individual chapters summarize specific findings regarding work presented at the end of each chapter. In what follows, the overall impacts of this work are summarized and placed in particular in the context of current design practice. In addition, recommendations for future work are provided.

10.1 Impacts on Design

A major finding from this research is that the maximum anchor load achieved during seismic inertial loading of NCSs anchored in cyclically cracked concrete was generally in the range of 100% to 120% $N_{um,cr}$ where $N_{um,cr}$ was the mean anchor ultimate tension load obtained from monotonic reference tests in statically open cracks for anchors that have passed seismic qualification testing (e.g. ACI 355.2, 2007). In this aspect, the experiments support the current code philosophy of using the reference anchor monotonic strength in cracked concrete with constant width cracks for seismic applications. The exception to this was the drop-in anchor, which failed in the dynamic tests at approximately 30 to 60% $N_{um,cr}$. This particular drop-in anchor has not been approved for seismic applications in cracked concrete, therefore its results should be judged apart from the other anchor types considered, which have all been approved for use in cracked concrete under a rigorous testing program in accordance with national standards.
The second major outcome, which is relevant to seismic design of anchors in practice, is the experimental documentation of the importance of anchor deformation capacity for seismic design of anchored nonstructural components. Specifically, for floor mounted, tension load cycling dominated anchored components, that remain elastic during a seismic event, there is a direct correlation between the mean anchor displacement capacity in cracked concrete $\delta_{um,cr}$ and the amplitude of seismic input motion that the anchored component system can withstand. This was demonstrated in the experiments, whereby the anchors that failed by concrete breakout, failed at input motion levels of approximately 60% amplitude scale whereas anchors of similar ultimate tension load capacity but having a pull-through failure failed at input motion levels of approximately 100% amplitude scale. This finding supports the current code “capacity design” philosophy for anchorage (ACI 318, 2008); that either the anchors should be “ductile” or the attachment of the anchor to the component should be designed as the yielding element for a yield force that is less than the anchor capacity, or if brittle anchors are used, the anchor should be designed for a load in excess of the expected demand with adequate factor of safety.

The third major impact of this work on design is the development of a lumped nonlinear hysteresis macro-model for simulating anchor tension load-displacement behavior in load cycling dominated applications in cracked concrete. The model uses a tension-only hook element located in series with a nonlinear element, thereby extending the application of the pivot hysteresis rule, developed by Dowell et al (1997) for reinforced concrete member behavior, to anchor modeling. It was observed from the experimental results that anchor behavior, as described by the envelope of the load-displacement hysteresis response due to seismic loading of NCSs anchored in cyclically cracked concrete generally can be captured using the mean monotonic load-displacement relationship for the corresponding static crack
width. Therefore, the model, which uses mean monotonic tension test data as input, is evaluated against tests conducted with expansion and undercut anchors having pull-through and concrete breakout failures, respectively. The nonlinear anchor model is then incorporated into a system model of an anchored NCS to predict maximum system response. Results from numerical simulations are compared with experimental results and recommendations are made for selecting model parameters that may be used to predict anchor load-displacement behavior and estimate maximum NCS response for tension load cycling dominated applications.

The results of this study as well as other recent studies (Hoehler and Eligehausen, 2008a and b and Hoehler et al., 2011a) suggest that anchor behavior subjected to dynamic rate cyclic loading as well as cyclic cracking boundary conditions can be reasonably estimated using the monotonic load-versus displacement behavior from reference testing in uncracked as well as cracked concrete where the cracks are open to a constant width. Given the need for knowledge of the mean monotonic load-displacement relationship for understanding anchor seismic behavior and thereby anchored system behavior, it would be useful if the mean monotonic load-displacement curve and estimates of the variability thereof are made available. This information can better assist the designer in selecting the proper anchors for seismic design. It is also useful for displacement-sensitive anchorage applications, where excessive anchor displacement may cause unacceptable NCS performance (e.g. anchorage of piping systems in which the pipe support deflection must remain limited). One possible avenue is to include the $N-\delta$ information in the Manufacture’s International Code Council Evaluation Services Reports (ICC-ESR) reports.
10.2 Future Research

Future research related to the seismic performance of nonstructural components anchored in concrete with cycled cracks should be considered. The following are of particular interest to advance understanding:

- The current test program addressed floor mounted NCSs. Future research should focus on the behavior of other NCS mounting configurations, such as wall-mounted and suspended systems.
- The current test program addressed anchors located in flexural/tension cracks due to lateral floor accelerations. NCSs mounted on floor slabs may also be sensitive to vertical accelerations. Since many NCSs may be very stiff in the vertical direction, it is possible that anchor loading would be more highly correlated with cyclic cracking of a slab under vertical seismic excitation. The effect of vertical accelerations should be investigated by analytical and experimental studies.
- The focus of this study was on flexural cracking of beams; however, shear cracking of slabs and shear walls will present different cracking patterns than studied herein. In particular, crack width amplitude, location and crack cycling characteristics.
- A series of in-phase load and crack width amplitude tests were performed in this research to represent the connections of structural components. The is a need for expansion of this work to determine if correlated load and crack width occurs in structural connections and the impact of such behavior on anchor load and other response parameters.
• Currently, seismic qualification testing of nonstructural components per ICC-ES AC156 (2010) or other test standards typically consider a component that is “hard-mounted” to the shake table, either via bolting or welding. Similar to seismic isolation-restraint devices, anchorage flexibility and inelastic actions will modify the component response and hence its demands. The nonlinear anchorage model, developed herein, could be implemented into system models of a broader variety of anchored nonstructural components in order to study the component response as modified by the anchor behavior.
Appendix A Model NCS Drawings & Calculations
GENERAL NOTES:

1) A total of 72 "weight plates" are required. See sheet 2 & 4.

2) Tolerances:
   2A) Member cuts shall be within +/- 1/10"
   2B) Bolt hole locations in the base tubes shall be within +/- 1/100"
       and shall be performed on a CNC milling machine
   2C) Bolt holes in the weight plates shall be within +/- 1/64"

MATERIAL PROPERTIES:

3) All tube shall be Hollow Structural Sections (HSS) "Tube Steel" - ASTM A 500 Gr. B, (UNO)

4) The 4"x4" Tube for Flexible system (Sheet 3) Shall be 100 ksi yield (min.) steel
   This tube may be fabricated as a built up section
   Material shall be: ASTM A 514 Gr. B (T1 Type A) or DOMEX 100 XF
   OR ENGINEERING APPROVED EQUIVALENT

5) Steel Plate and angle shall conform to ASTM A 36 (min.)
   Alternate cost savings material proposals for the weight plates will be considered provided
   the weight of the plates is similar or greater

6) Threaded rod shall conform to ASTM A 193 B7 (UNO) - PROVIDED BY OTHERS

7) All welds shall be E70XX electrode (min.)

UNO = Unless Noted Otherwise

REVISION: REASON FOR REVISION:

1  Revise Sheet 14 hole size
2  Revise Sheets 1 & 6 as noted, Add Sheet 10A
3  Revise Sheets 9 & 13 as noted
4  Revise Sheets 11 &

For questions please contact Derrick Watkins at dwatkins@ucsd.edu or 858-997-8885
**ASSEMBLY A: Rigid Mast Assembly**

**DELETED**

---

**ASSEMBLY B: Flexible Mast Assembly**

<table>
<thead>
<tr>
<th>DESCRIPTION</th>
<th>PART NAME</th>
<th>Qty.</th>
</tr>
</thead>
<tbody>
<tr>
<td>B1 High Yield Strength Tube</td>
<td>Mast</td>
<td>1</td>
</tr>
<tr>
<td>B2 HSS</td>
<td>Base Tube</td>
<td>1</td>
</tr>
<tr>
<td>B3 HSS</td>
<td>Tube Spacer</td>
<td>6</td>
</tr>
<tr>
<td>B4 Plate</td>
<td>Tab</td>
<td>4</td>
</tr>
<tr>
<td>B5 Plate</td>
<td>Side Stiffener</td>
<td>4</td>
</tr>
<tr>
<td>B6 Plate</td>
<td>Front/Back Stiffener</td>
<td>2</td>
</tr>
<tr>
<td>B7 Plate</td>
<td>Cap Plate</td>
<td>1</td>
</tr>
<tr>
<td>B8 Round Tube</td>
<td>Guide Post</td>
<td>6</td>
</tr>
<tr>
<td>B9 Angle</td>
<td>Diagonal Brace</td>
<td>2</td>
</tr>
</tbody>
</table>
### ASSEMBLY C: Shear Sled

<table>
<thead>
<tr>
<th>PART NAME</th>
<th>Qty.</th>
</tr>
</thead>
<tbody>
<tr>
<td>C1 Bottom Plate</td>
<td>1</td>
</tr>
<tr>
<td>C2 End Tube</td>
<td>2</td>
</tr>
<tr>
<td>C3 Side Tube</td>
<td>2</td>
</tr>
<tr>
<td>C4 Stiffener Plate</td>
<td>2</td>
</tr>
<tr>
<td>C5 Bottom Bar</td>
<td>2</td>
</tr>
<tr>
<td>C6 Top Bar</td>
<td>2</td>
</tr>
</tbody>
</table>

### ASSEMBLY D: Weight Plates

<table>
<thead>
<tr>
<th>PART NAME</th>
<th>Qty.</th>
</tr>
</thead>
<tbody>
<tr>
<td>D1 Weight Plate</td>
<td>72</td>
</tr>
</tbody>
</table>
See Note 2 on Sheet 1 for Hole Tolerances
Measure holes from centerline

B1
4x4x3/8" 100 ksi yield strength Tube
(See Note 4, Sheet 2)

B2
6x6x1/2" HSS

SEISMIC ANCHORAGE PROJECT
DETAILS
By: DAW CHI TH Date: 09/07/00 Scale: NTS Sheet: 7
11/16" Holes
2 PLCS

1-1/2" OD Round Tube
1/4" Wall Thickness

2x2x3/8" Angle
4'-2" long
2'-10"

1/2" thick plate

C1
See Note 2 on Sheet 1 for Hole Tolerances

Measure Holes from Centerline & bottom of tube
C4

\( \frac{1}{2} \) " Thick Plate
(2) 1\(\frac{3}{4}\)" Dia. Thru Holes

(4) \(\frac{7}{8}\)" Dia. Thru Holes

D1

1" Thick Plate
**WALLE Calculations:**

**Calculation of center-of-mass (flexible WALLE)**

- 2035 lbf @ 62.5 in. (weight plates)
- Measured mass: 54 plates × 37.68 lbf = 2035 lbf
- 90.8 lbf @ 39.5 in. (mast tube)
  
  \[ A = 4.78 \text{ in.}^2 \]
  
  \[ 67 \text{ in.} \times 4.78 \text{ in.}^2 \times 0.283 \text{ lbf/in.}^3 = 90.8 \text{ lbf} \]
- 159.2 lbf @ 3 in. (base tube)
  
  \[ A = 11.7 \text{ in.}^2 \]
  
  \[ 48 \text{ in.} \times 11.7 \text{ in.}^2 \times 0.283 \text{ lbf/in.}^3 = 159.2 \text{ lbf} \]
- 100 lbf @ 3 in. (SAMU)
  
  Approximate mass: 25 lbf per SAMU

\[ M_{1-4} = 2035 + 90.8 + 159.2 + 100 = 2385 \text{ lbf} \]

\[ H_{CM} = \frac{(2035 \times 62.5 + 90.8 \times 39.5 + 159.2 \times 3 + 100 \times 3)}{2385} = 55.2 \text{ in} \]

Use \( H_{CM} = 55 \text{ in.} \).

WALLE weight as measured in lab: **2550 lbf** (includes thread rods, stiffeners, cap PL, lifting eyes, & angle braces)

**Calculation of center-of-mass (rigid WALLE)**

- 2035 lbf @ 62.5 in. (weight plates)
- Measured mass: 8 plates × 37.68 lbf = 2035 lbf
- 90.8 lbf @ 39.5 in. (mast tube)
  
  \[ A = 4.78 \text{ in.}^2 \]
  
  \[ 67 \text{ in.} \times 4.78 \text{ in.}^2 \times 0.283 \text{ lbf/in.}^3 = 90.8 \text{ lbf} \]
- 159.2 lbf @ 3 in. (base tube)
  
  \[ A = 11.7 \text{ in.}^2 \]
  
  \[ 48 \text{ in.} \times 11.7 \text{ in.}^2 \times 0.283 \text{ lbf/in.}^3 = 159.2 \text{ lbf} \]
- 100 lbf @ 3 in. (SAMU)
  
  Approximate mass: 25 lbf per SAMU

\[ M_{1-4} = 301.4 + 90.8 + 159.2 + 100 = 651 \text{ lbf} \]

\[ H_{CM} = \frac{(301.4 \times 62.5 + 90.8 \times 39.5 + 159.2 \times 3 + 100 \times 3)}{651} = 35.6 \text{ in} \]

Use \( H_{CM} = 38 \text{ in.} \).

WALLE weight as measured in lab: **860 lbf** (includes thread rods, stiffeners, cap PL, lifting eyes, & angle braces)
Conventions for correction of WALLE displacement data

**String pots on WALLE mast**

*Horizontal displacements measured on the WALLE mast will be projected to the calculated center-of-mass @ 55 in. above the slab.*

\[ H_{SpL} = 48 \text{ in. (measured height of longitudinal string pot above slab)} \]

\[ H_{SpT} = 51.5 \text{ in. (measured height of transverse string pot above slab)} \]

\[ \Delta_{SpL} = \text{measured longitudinal WALLE displacement on mast (relative to slab)} \]

\[ \Delta_{SpT} = \text{measured transverse WALLE displacement on mast (relative to slab)} \]

Longitudinal:

- Correction for rigid body rotation between the \( H_{SpL} \) and \( H_{CM} \)

\[
\Delta_{CML} = \frac{H_{CM}}{H_{SpL}} \Delta_{SpL} = 1.14
\]

- Correction for string elongation due to vertical lifting of the mast \( \rightarrow \) Neglect

Induced error calculated for worst case (8.5 in. string pot length) to be \(~0.018\) inches (0.46 mm), which is less than the published sensitivity of the string pot (0.15% @25 in. = 0.0375 in. [0.95 mm]). In most cases the string length was much longer.
• Correction for elastic bending of mast between the $H_{SpL}$ and $H_{CM}$

$$HSS\ 4\times\ 4\times\ 3/8\ in.$$  
$$E = 29,000\ ksi;\ I_x = 10.3\ in.^4$$  
$$F_{CM,max} = 20\ kip\times\ 15.5\ in./55\ in. = 5.6\ kip$$  

$$v = \frac{F_{CM,max} x^2 (3H_{CM} - x)}{6EI}$$  
$$v(x = H_{CM}) = 1.04\ in.$$  
$$v(x = H_{SpL}) = 0.84\ in.$$  

$$\Delta_{CMLb} = \frac{2H_{CM}^3}{H_{SpL}^2 (3H_{CM} - H_{SpL})} \Delta_{SpL} = 1.23$$

• Correction for elastic shear deformation of mast between the $H_{SpL}$ and $H_{CM}$, WALLE mast is tall and slender $\rightarrow$ Neglect shear deformations

1. The correction for rigid body motion is theoretically applied only to the displacement due to from the “nonlinear” behavior (mostly from the anchors).

2. The correction for bending is theoretically applied only to the elastic bending component of the displacement.

$$v_{el}(@\ H_{SpL}) = \frac{F_{CM} H_{SpL}^2 (3H_{CM} - H_{SpL})}{6EI}$$  
$$v_{nl}(@\ H_{SpL}) = \Delta_{SpL} - v_{el}$$  

$H_{CM} =$ calculated center-of-mass for WALLE  
$F_{CM} =$ measured force (weight plate mass $\times$ accl.) at WALLE center-of-mass

$$\Delta_{CML} = \frac{2H_{CM}^3}{H_{SpL}^2 (3H_{CM} - H_{SpL})} v_{el} + H_{CM} \frac{H_{CM}}{H_{SpL}} v_{el}$$

Or ignoring bending...

$$\Delta_{CML} = H_{CM} H_{SpL} \Delta_{SpL} = 1.146 \Delta_{SpL}$$
Transverse:

- Correction for rigid body rotation

\[
\Delta_{\text{SpTr}} = \sqrt{\left(\Delta_{\text{SpT}} + L_{\text{SpT}}\right)^2 - \Delta_{\text{Spl}}^2}
\]

- Correction for string elongation due to vertical lifting of the mast \(\rightarrow\) Neglect

- Correction for elastic bending of mast between the \(H_{\text{Spl}}\) and \(H_{\text{CML}}\) \(\rightarrow\) Neglect
  (Transverse loads are small)

- Correction for elastic shear deformation of mast between the \(H_{\text{Spl}}\) and \(H_{\text{CML}}\) \(\rightarrow\) Neglect

\[
\Delta_{\text{CM}} = \Delta_{\text{SpTr}} = \sqrt{\left(\Delta_{\text{SpT}} + L_{\text{SpT}}\right)^2 - \Delta_{\text{Spl}}^2 - L_{\text{SpT}}}
\]
Appendix B Cyclic Crack Inertial Loading Rig (CCILR) Drawings
# UCSD Cyclic Crack Test Frame

## Parts List

This parts list contains parts to be furnished as part of the "Cyclic Crack Test Frame" package. Items marked by "OTHERS" are not part of this package, but are necessary for the testing program.

<table>
<thead>
<tr>
<th>DETAIL DWG.</th>
<th>FURNISHED BY</th>
<th>PART NO.</th>
<th>PART</th>
<th>DESCRIPTION</th>
<th>QUANTITY</th>
</tr>
</thead>
<tbody>
<tr>
<td>29</td>
<td>Rossin</td>
<td>A1</td>
<td>End Beams</td>
<td>W14x176</td>
<td>2</td>
</tr>
<tr>
<td>33</td>
<td>Rossin</td>
<td>A2</td>
<td>Stiffeners - End Beams</td>
<td>3/4&quot; Thick Steel Plate</td>
<td>44</td>
</tr>
<tr>
<td>---</td>
<td>Rossin</td>
<td>A3</td>
<td>DELETED</td>
<td>DELETED</td>
<td>DELETED</td>
</tr>
<tr>
<td>31</td>
<td>Rossin</td>
<td>A4</td>
<td>Fixed Base Plate</td>
<td>1 3/8&quot; Thick Steel Plate</td>
<td>2</td>
</tr>
<tr>
<td>31</td>
<td>Rossin</td>
<td>A5</td>
<td>Sliding Baseplate</td>
<td>1&quot; Thick Steel Plate</td>
<td>2</td>
</tr>
<tr>
<td>33</td>
<td>Rossin</td>
<td>A6</td>
<td>Stiffeners - Slab hold downs</td>
<td>3/4&quot; Thick Steel Plate</td>
<td>4</td>
</tr>
<tr>
<td>17</td>
<td>Rossin</td>
<td>B1-L</td>
<td>Bottom Beam</td>
<td>W12x45</td>
<td>1</td>
</tr>
<tr>
<td>19</td>
<td>Rossin</td>
<td>B1-R</td>
<td>Bottom Beam</td>
<td>W12x45</td>
<td>1</td>
</tr>
<tr>
<td>21/22</td>
<td>Rossin</td>
<td>B2</td>
<td>Cross Beam (ends coped)</td>
<td>W12x45</td>
<td>2</td>
</tr>
<tr>
<td>26</td>
<td>Rossin</td>
<td>B3</td>
<td>Stopper Angle</td>
<td>L 3.5 x 3.5 x 3/8</td>
<td>16</td>
</tr>
<tr>
<td>---</td>
<td>Rossin</td>
<td>B4</td>
<td>DELETED</td>
<td>DELETED</td>
<td>DELETED</td>
</tr>
<tr>
<td>---</td>
<td>Rossin</td>
<td>B5</td>
<td>DELETED</td>
<td>DELETED</td>
<td>DELETED</td>
</tr>
<tr>
<td>25</td>
<td>Rossin</td>
<td>B6</td>
<td>Stiffener - Bottom Beam</td>
<td>1/2&quot; Thick Steel Plate</td>
<td>24</td>
</tr>
<tr>
<td>24</td>
<td>Rossin</td>
<td>B7</td>
<td>Stiffener - Bottom Beam (with 3 holes)</td>
<td>1/2&quot; Thick Steel Plate</td>
<td>4</td>
</tr>
<tr>
<td>28</td>
<td>Rossin</td>
<td>B8</td>
<td>Slab Support Roller</td>
<td>Round HSS 4&quot; x 0.25&quot;</td>
<td>4</td>
</tr>
<tr>
<td>28</td>
<td>Rossin</td>
<td>B9</td>
<td>Posts, 1 end threaded, other end smooth</td>
<td>1&quot; Dia round bar</td>
<td>4</td>
</tr>
<tr>
<td>28</td>
<td>Rossin</td>
<td>B10</td>
<td>Nuts for 1&quot; Dia threaded post</td>
<td>Heavy Hex Nut</td>
<td>8</td>
</tr>
<tr>
<td>---</td>
<td>?? Rossin ??</td>
<td>C1</td>
<td>Bolts for (B3 to B1) length as required</td>
<td>1/2&quot; Dia. Bolts (≤ 2.5&quot; long)</td>
<td>32</td>
</tr>
<tr>
<td>27</td>
<td>?? Rossin ??</td>
<td>C2</td>
<td>Bolts for (B1 to A4) length as required</td>
<td>1&quot; Dia A 490 Bolts (≤ 3&quot; long)</td>
<td>8</td>
</tr>
<tr>
<td>27</td>
<td>?? Rossin ??</td>
<td>C3</td>
<td>Bolts for (B2 to B5) length as required</td>
<td>1&quot; Dia A 490 Bolts (≤ 2&quot; long)</td>
<td>12</td>
</tr>
<tr>
<td>---</td>
<td>OTHERS</td>
<td>C4</td>
<td>DYW/DAG Bars (for frame compression)</td>
<td>7/8&quot; &quot;Threadbar&quot; Form Tie</td>
<td>4</td>
</tr>
<tr>
<td>---</td>
<td>OTHERS</td>
<td>C5</td>
<td>Actuator Bolts</td>
<td>1.25&quot; Dia. HS Thread Rod</td>
<td>16</td>
</tr>
<tr>
<td>---</td>
<td>OTHERS</td>
<td>D1</td>
<td>Teflon Sliding Pads</td>
<td>1/4&quot; Thick Sheet</td>
<td>2</td>
</tr>
<tr>
<td>---</td>
<td>OTHERS</td>
<td>D2</td>
<td>Teflon/Neoprene Bearing Pads (for B8)</td>
<td>0.21&quot; Thick Sheet</td>
<td>8</td>
</tr>
<tr>
<td>32</td>
<td>Rossin</td>
<td>D3</td>
<td>Coki Rolled Stainless Steel Sliding Pad</td>
<td>1/8&quot; Thick Sheet</td>
<td>2</td>
</tr>
<tr>
<td>15</td>
<td>Rossin</td>
<td>E1</td>
<td>Crack Inducer (TYPE I) sheet 15</td>
<td>12 GA Galvanized Sheet</td>
<td>9</td>
</tr>
<tr>
<td>16</td>
<td>Rossin</td>
<td>E2</td>
<td>Crack Inducer (TYPE U) sheet 16</td>
<td>12 GA Galvanized Sheet</td>
<td>2</td>
</tr>
</tbody>
</table>
**GENERAL NOTES:**

1) All dimensions are in "inches", Unless Noted Otherwise (UNO)

2) Tolerances:
   2A) Member cuts shall be within +/- 1/10"
   2B) Bolt hole locations shall be within +/- 1/84"

3) All tube shall be Hollow Structural Sections (HSS) "Tube Steel" - ASTM A 600 Gr. B, (UNO)

4) Steel Plate and angle shall conform to ASTM A 36 minimum (UNO)

5) Threaded rod shall conform to ASTM A 193 B7 (UNO)

6) All welds shall be E70XX electrode (min.)

7) Fabricator to furnish all items except as listed "BY OTHERS"
   See detailed parts list for all items to be furnished by fabricator

8) All bolts shall be ASTM A490, Unless Noted Otherwise (UNO)

---

**REVISION:**

<table>
<thead>
<tr>
<th>DATE</th>
<th>REASON FOR REVISION</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 12/05/08</td>
<td>DRAFT FOR COST ESTIMATE (NOT FOR FABRICATION)</td>
</tr>
<tr>
<td>2 01/05/08</td>
<td>ISSUE FOR CONSTRUCTION</td>
</tr>
<tr>
<td>3 01/20/09</td>
<td>REVISED SHEETS 28 &amp; 32 PER FABRICATOR REQUEST FOR INFORMATION (RFI)</td>
</tr>
</tbody>
</table>

For questions please contact Derrick Watkins at dwatkins@ucsd.edu or 858-997-8885

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**Cyclic Crack Test Frame**

**TEST FRAME - GENERAL NOTES**

**BY:** DW\_UNAL  **CH\_IC**  **TH**  **DATE:** 12/01/08  **SCALE:** NTS  **SHEET:** 1/33
Section A - Outside View - Longitudinal Beam

Cyclic Crack Test Frame

SIDE VIEW - BOTTOM FRAME

BY: DWJWAL CHIC TH DATE: 12/31/06 SCALE: NTS SHEET: 4/33
Section 1

Section 2

Section 3

Cyclic Crack Test Frame

END VIEW - BASE FRAME SECTIONS

BY: DWUNA
CHIC TH
DATE: 12/31/08 SCALE: NTS SHEET: 633
Cyclic Crack Test Frame

Section C  Section A
Section 1 (Dimensions)

Section 1 (Part Callout)
Detail 4
(Fixed End)
Plan View

Cyclic Crack Test Frame
Part B1-R W12x45 Cutting Plan

BY: DWUNAL CHK: TH DATE: 12/31/06 SCALE: NTS SHEET: 1033
Cyclic Crack Test Frame
Part B1-R W12x45 Assembly Plan

BY: DWU0104
CHK: TH
DATE: 12/31/08
SCALE: NTS
SHEET: 2003
Cyclic Crack Test Frame

B2 W12x45 Cutting Plan (Bottom View, Looking up)

BY: [Signature] CHK: [Signature] TH: [Signature] DATE: 12/31/08 SCALE: NT8 SHEET: 22/33
B7 $\frac{1}{2}''$ Thick Steel Plate (4 PLCS)

3 - 17/16" Dia. Holes for 1" Dia. A490 Bolts (TYP)

Note: Stiffener Dimensions already include a $\frac{1}{16}''$ gap between stiffener and connecting W-Shape.
B6 $\frac{1}{2}$" Thick Steel Plate
(24 PLCS)

Note: Stiffener Dimensions already include a $\frac{1}{16}$" gap between stiffener and connecting W-Shape
B3 - Top View

B3 L3.5"x3.5"x3/8 (16 PLCS)

9/16" Dia. Thru Holes

1 1/4" 1 1/4" 1 1/2"

3 1/2" 1 15/16"

B3 L3.5"x3.5"x3/8 Cutting Plan

BY: DWJ/NAL CHK: TH DATE: 12/01/08 SCALE: NT8 SHEET: 28/33
Cyclic Crack Test Frame

C2 and C3 A490 Bolts

UCSD

BY: DWJNAL CHK TH DATE: 12/1/08 SCALE: N78 SHEET: 27/33
Side View @ A6

Front View Assembly  Side View @ A2

Cutting Plan  (Top & Bottom Flange)

A1 (2 TOTAL)

Cyclic Crack Test Frame

Part A1 Details

BY: DWNIAL  CHK: TH  DATE: 12/31/08  SCALE: NTS  SHEET: 29/03

UCSD
A1 (Sliding End)

A1 (Fixed End)

Assembly Drawing (Top View)
A5 (2 Total)
1" Thick Plate

A4 (2 Total)
1-3/8" Thick Plate
D3 (2 Total)
1/8" Thick
Cold Rolled S.S.

D3 to be Epoxy glued to B1-L & B1-R

Cyclic Crack Test Frame
Part D3

UCSD

BY: DWUNAL   CHK: TH   DATE: 01/25/09   SCALE: NAT   SHEET: 32/33
Note: Stiffener dimensions shown include a $\frac{1}{16}$" gap between stiffener and connecting part

A2 (44 Total)
3/4" Thick Plate

A6 (4 Total)
3/4" Thick Plate

Cyclic Crack Test Frame
Part Details A2 & A6

BY: DWUNIAL CHK TH DATE: 12/31/06 SCALE: NT8 SHEET: 33/33
**Test description**
Dynamic tests of the WALLE in cracked concrete.

**Anchor & S/N**
N4 - Epoxy
1/2" Dia. B7 Rod, 5.15" long

**Load / Wcr History**
FM02 / CR02 (0.8mm max. crack width)

### Installation parameters

<table>
<thead>
<tr>
<th>Concrete specimen</th>
<th>Low-strength Vulcan 202502CD</th>
</tr>
</thead>
<tbody>
<tr>
<td>Date cast</td>
<td>08/05/09</td>
</tr>
<tr>
<td>Compression strength</td>
<td>3798 psi / 26.2 MPa (64 day on 10/08/2009)</td>
</tr>
<tr>
<td>Target crack opening (in.)</td>
<td>0.8 mm (0.0315&quot;)</td>
</tr>
<tr>
<td>Drill type</td>
<td>Hilti TE40-ARV</td>
</tr>
<tr>
<td>Drill bit Ø dcut (in.)</td>
<td>0.584&quot;</td>
</tr>
<tr>
<td>Drill hole depth h1 (in.)</td>
<td>2.4&quot;</td>
</tr>
<tr>
<td>Effective depth hef (in.)</td>
<td>2.4&quot;</td>
</tr>
<tr>
<td>Drill hole cleaning</td>
<td>3xblow, 3xbrush, 3xblow, 3xbrush, 3xblow</td>
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<tr>
<td>Installation torque</td>
<td>n/a</td>
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### Instrumentation

<table>
<thead>
<tr>
<th>Crack Width Pots</th>
<th>SAMU 1 (A-D)</th>
<th>S18, S2, S3, S4</th>
</tr>
</thead>
<tbody>
<tr>
<td>SAMU 2 (A-D)</td>
<td>S5, S6, S7, S8</td>
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</tr>
<tr>
<td>SAMU 3 (A-D)</td>
<td>S9, S10, S11, S12</td>
<td></td>
</tr>
<tr>
<td>SAMU 4 (A-D)</td>
<td>S13, S14, S15, S16</td>
<td></td>
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<tr>
<td>Disp N1-N4</td>
<td>119587, 119030, 119032, 122033</td>
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<tr>
<td>Disp V1-V2</td>
<td>122875, 122879</td>
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<tr>
<td>Disp Long, Disp Tran</td>
<td>204858, 0003-11971</td>
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<tr>
<td>Accl</td>
<td>960, 1386, 1733, 1734, 1735, 1736, 1737, 1738</td>
<td></td>
</tr>
<tr>
<td>Rod 1-4</td>
<td>R5, R2, R3, R4</td>
<td></td>
</tr>
</tbody>
</table>

### Anchor

| Height above concrete (in.) | 2.66 | 2.78 | 2.75 | 2.67 |
| Hairline crack (mm) | 0.10 | 0.08 | 0.08 | 0.05 |
| Crack opening Δw (mm) | 0.8 | 0.8 | 0.8 | 0.8 |
| Anchor preload (lb) | 50 lbs target (50-70 lbs OK range) |
| Anchor disp. during set (in) | n/a | n/a | n/a | n/a |
| Failure Mode | C | n/a | C | n/a |
| Rmax (in.) | 12.5 | 12 | 9.5 | 9.5 |
| Rmin (in.) | 10.5 | 10 | 7 | 6.5 |
| h (in.) | 2.25 | 2.25 | 2.25 | 7 |

**Comments:**
- LP0/1 became unattached during the failure b/c it was within the field of the breakout cone of anchor 1

**Key**
- C = Concrete failure
- S = Steel failure
- Po = Pullout (including sleeve)
- Pt = Pull-through
- Sp = Splitting
**TEST MOTION PARAMETERS**

<table>
<thead>
<tr>
<th>Test name</th>
<th>Anchor</th>
<th>Type</th>
<th>Crack Width</th>
<th>Table Loading</th>
<th>Crack Record</th>
<th>Motion</th>
<th>Bldg</th>
<th>Floor</th>
<th>Target Scale (%)</th>
<th>PIA (g)</th>
<th>PID (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>WALLE_F_N5_Hammer_1000lb</td>
<td>N4</td>
<td>Epoxy</td>
<td>0.0mm</td>
<td>n/a</td>
<td>--</td>
<td>Hammer</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>WALLE_F_N4_UC_WN</td>
<td>N4</td>
<td>Epoxy</td>
<td>0.0mm</td>
<td>0.05WN</td>
<td>n/a</td>
<td>White Noise</td>
<td>n/a</td>
<td>n/a</td>
<td>100</td>
<td>0.03</td>
<td></td>
</tr>
<tr>
<td>WALLE_F_N4_CC08_WN</td>
<td>N4</td>
<td>Epoxy</td>
<td>0.8mm</td>
<td>0.05WN</td>
<td>CR02_08</td>
<td>White Noise</td>
<td>n/a</td>
<td>n/a</td>
<td>100</td>
<td>0.03</td>
<td></td>
</tr>
<tr>
<td>WALLE_F_N4_CC08_FM02_5P</td>
<td>N4</td>
<td>Epoxy</td>
<td>0.8mm</td>
<td>FM02</td>
<td>CR02_08</td>
<td>kobe00</td>
<td>2st</td>
<td>1</td>
<td>5</td>
<td>0.05</td>
<td>0.2</td>
</tr>
<tr>
<td>WALLE_F_N4_CC08_FM02_15P</td>
<td>N4</td>
<td>Epoxy</td>
<td>0.8mm</td>
<td>FM02</td>
<td>CR02_08</td>
<td>kobe00</td>
<td>2st</td>
<td>1</td>
<td>15</td>
<td>0.14</td>
<td>0.6</td>
</tr>
<tr>
<td>WALLE_F_N4_CC08_FM02_20P</td>
<td>N4</td>
<td>Epoxy</td>
<td>0.8mm</td>
<td>FM02</td>
<td>CR02_08</td>
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<td>2st</td>
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<td>20</td>
<td>0.19</td>
<td>0.7</td>
</tr>
<tr>
<td>WALLE_F_N4_CC08_FM02_30P</td>
<td>N4</td>
<td>Epoxy</td>
<td>0.8mm</td>
<td>FM02</td>
<td>CR02_08</td>
<td>kobe00</td>
<td>2st</td>
<td>1</td>
<td>30</td>
<td>0.29</td>
<td>1.1</td>
</tr>
</tbody>
</table>

**TEST SETUP**

**DETAILED ANCHOR FAILURE PHOTOS**

- **Anchor 4 (N-W)**
- **Anchor 2 (N-E)**
- **Anchor 3 (S-W)**
- **Anchor 1 (S-E)**

**SIDE VIEW**

**PLAN VIEW FAILURE PHOTO**
Test was conducted at SDSU as part of uncracked concrete test program, data included for comparison with cracked concrete case.
Test was conducted at SDSU as part of uncracked concrete test program, data included for comparison with cracked concrete case. South anchor load and displacement are plotted negative for clarity, the load is tension and displacement is up.
Test was conducted at SDSU as part of uncracked concrete test program, data included for comparison with cracked concrete case.
Test was conducted at SDSU as part of uncracked concrete test program, data included for comparison with cracked concrete case. South anchor load and displacement are plotted negative for clarity, the load is tension and displacement is up.
Max Accel = 0.69 g

Max Disp = 0.58 in

Max WALLE Accel = 1.12 g

Max WALLE Disp = 0.66 in
South anchor load and displacement are plotted negative for clarity, although, the load is tension and displacement is up.
**WALLE_F_N4_CC08_FM02_30P**

**Anchor Type: Epoxy**

**Slab Accel**

Abs. Accel.

Max Accel = 0.85 g

**Slab Disp**

Abs. Disp.

Max Disp = 0.87 in

**WALLE Accel**

Abs. Accel. @ WALLE CM

Max WALLE Accel = 0.98 g

**WALLE Disp**

Abs. Disp. @ WALLE CM

Max WALLE Disp = 2.55 in

**Time, sec**

**Spectral Accel, g**

Period, sec

2% Damping

**WALLE Drift Ratio, %**

WALLE Accel, g

WALLE Drift Ratio, %
## Hill Seismic Project - WALLE Dynamic Data Sheet

### Test description
Dynamic tests of the WALLE in cracked concrete.

<table>
<thead>
<tr>
<th>Anchor &amp; S/N</th>
<th>N5 - Expansion</th>
</tr>
</thead>
</table>

| Load/Wc History | FM02/CR02 (0.5mm max. crack width) |

### Series
WALLE_F_N5_CC05_FM02

### Date
12/9/2009

### Engineer(s)
Derrick Watkins, Arnold Gastelum
Allison Yu, Yujia Liu

### Technician(s)
Paul Grecco

---

### Installation parameters

<table>
<thead>
<tr>
<th>Concrete specimen</th>
<th>Low-strength Vulcan 202502CD</th>
</tr>
</thead>
<tbody>
<tr>
<td>Date cast</td>
<td>08/05/09</td>
</tr>
<tr>
<td>Compression strength</td>
<td>3798 psi / 26.2 MPa (64 day on 10/08/2009)</td>
</tr>
<tr>
<td>Target crack opening (in.)</td>
<td>0.5 mm (0.0197&quot;)</td>
</tr>
<tr>
<td>Drill type</td>
<td>Hilti TE40-ARV</td>
</tr>
<tr>
<td>Drill bit Ø dcut (in.)</td>
<td>0.5&quot; (Recommended) 0.515&quot; (Measured)</td>
</tr>
<tr>
<td>Drill hole depth h1 (in.)</td>
<td>4.125&quot;</td>
</tr>
<tr>
<td>Effective depth hef (in.)</td>
<td>3.75&quot;</td>
</tr>
<tr>
<td>Drill hole cleaning</td>
<td>3xblow, 3xblow, 3xblow</td>
</tr>
<tr>
<td>Installation torque</td>
<td>40 ft-lb, then release to 20 ft-lb</td>
</tr>
<tr>
<td>Slab/NCS position</td>
<td>Slab 4, Position B</td>
</tr>
<tr>
<td>Data sample rate (Hz)</td>
<td>200 Hz</td>
</tr>
</tbody>
</table>

### Installation instrumentation

<table>
<thead>
<tr>
<th>Table, Act E&amp;W P&amp;A</th>
<th>Crack Width Pots</th>
</tr>
</thead>
<tbody>
<tr>
<td>LP01, LP02, LP03, LP04</td>
<td></td>
</tr>
<tr>
<td>SAMU 1 (A-D)</td>
<td>S18, S2, S3, S4</td>
</tr>
<tr>
<td>SAMU 2 (A-D)</td>
<td>S5, S6, S7, S8</td>
</tr>
<tr>
<td>SAMU 3 (A-D)</td>
<td>S9, S10, S11, S12</td>
</tr>
<tr>
<td>SAMU 4 (A-D)</td>
<td>S13, S14, S15, S16</td>
</tr>
<tr>
<td>Disp N1-N4</td>
<td>119587, 119030, 119032, 122033</td>
</tr>
<tr>
<td>Disp V1-V2</td>
<td>122875, 122879</td>
</tr>
<tr>
<td>Disp Long, Disp Tran</td>
<td>20498, 0003-11971</td>
</tr>
<tr>
<td>LP01, LP02, LP03, LP04</td>
<td></td>
</tr>
<tr>
<td>S18, S2, S3, S4</td>
<td></td>
</tr>
<tr>
<td>LW1-LW4</td>
<td>229931, 239833, 239634, 239635</td>
</tr>
</tbody>
</table>

### Anchor

| Anchor final disp. (in.) | 1.33 |

### Loading/Setup

<table>
<thead>
<tr>
<th>Anchors</th>
<th>SE (1)</th>
<th>NE (2)</th>
<th>SW (3)</th>
<th>NW (4)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.04</td>
<td>4.07</td>
<td>4.05</td>
<td>4.08</td>
<td></td>
</tr>
</tbody>
</table>

| Height abv. conc. (in.) post | 3.12 | 3.10 | 3.13 | 3.07 |
| Hairline crack (mm) | 0.10 | 0.08 | 0.10 | 0.08 |
| Crack opening Δw (mm) | 0.5 | 0.5 | 0.5 | 0.5 |
| Anchor preload | 20 ft-lb (~1,500 lbs) |
| Height abv. conc. (in.) pre | 2.97 | 2.97 | 2.96 | 2.9 |
| Anchor disp. during set (in) | 0.15 | 0.13 | 0.17 | 0.17 |
| Failure Mode | Pt | n/a | Pt | n/a |
| Rmax (in.) | n/a | n/a | n/a | n/a |
| Rmin (in.) | n/a | n/a | n/a | n/a |
| h (in.) | n/a | n/a | n/a | n/a |

**Key**
- C = Concrete failure
- S = Steel failure
- Po = Pullout (including sleeve)
- Pt = Pull-through
- Sp = Splitting

---

**Comments:**
### TEST MOTION PARAMETERS

<table>
<thead>
<tr>
<th>Test name</th>
<th>Anchor</th>
<th>Type</th>
<th>Crack Width</th>
<th>Table Loading</th>
<th>Crack Record</th>
<th>Motion</th>
<th>Motion Scale (%)</th>
<th>Scaled PIA (g)</th>
<th>Scaled PID (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>WALLE_F_N5_Hammer_1000lb</td>
<td>N5</td>
<td>Expansion</td>
<td>0.0mm</td>
<td>n/a</td>
<td>--</td>
<td>Hammer</td>
<td>--</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>WALLE_F_N5_UC_WN</td>
<td>N5</td>
<td>Expansion</td>
<td>0.0mm</td>
<td>0.05WN</td>
<td>CR02_05</td>
<td>White Noise</td>
<td>n/a</td>
<td>n/a</td>
<td>100</td>
</tr>
<tr>
<td>WALLE_F_N5_CC05_WN</td>
<td>N5</td>
<td>Expansion</td>
<td>0.5mm</td>
<td>0.05WN</td>
<td>CR02_05</td>
<td>White Noise</td>
<td>n/a</td>
<td>n/a</td>
<td>100</td>
</tr>
<tr>
<td>WALLE_F_N5_CC05_FM02_10P</td>
<td>N5</td>
<td>Expansion</td>
<td>0.5mm</td>
<td>FM02</td>
<td>CR02_05</td>
<td>Kobe00</td>
<td>2st</td>
<td>1</td>
<td>10</td>
</tr>
<tr>
<td>WALLE_F_N5_CC05_FM02_30P</td>
<td>N5</td>
<td>Expansion</td>
<td>0.5mm</td>
<td>FM02</td>
<td>CR02_05</td>
<td>Kobe00</td>
<td>2st</td>
<td>1</td>
<td>30</td>
</tr>
<tr>
<td>WALLE_F_N5_CC05_FM02_100P</td>
<td>N5</td>
<td>Expansion</td>
<td>0.5mm</td>
<td>FM02</td>
<td>CR02_05</td>
<td>Kobe00</td>
<td>2st</td>
<td>1</td>
<td>100</td>
</tr>
<tr>
<td>WALLE_F_N5_CC05_FM02_100Pa</td>
<td>N5</td>
<td>Expansion</td>
<td>0.5mm</td>
<td>FM02</td>
<td>CR02_05</td>
<td>Kobe00</td>
<td>2st</td>
<td>1</td>
<td>100</td>
</tr>
</tbody>
</table>

### TEST SETUP

- Anchor 4 (N-W)
- Anchor 2 (N-E)
- Anchor 3 (S-W)
- Anchor 1 (S-E)

### SIDE VIEW

- PLAN VIEW FAILURE PHOTO

- NW
- SW
- NE
- SE
Test was inadvertently halted at approximately 5 seconds due to a tripped limit on the shake table controller.
WALLE_F_N5_CC05_FM02_100Pa

Anchor Type: Expansion

Max Accel = 1.53 g

Max Disp = 2.88 in

Max WALLE Accel = 1.87 g

Max WALLE Disp = 5.39 in
## Hilti Seismic Project - WALLE Dynamic Data Sheet

### Test description

Dynamic tests of the WALLE in cracked concrete.

### Anchor & S/N

N5 - Expansion anchor

### Load History

FM02 / CR02

### Installation parameters

<table>
<thead>
<tr>
<th>Concrete specimen</th>
<th>Low-strength Vulcan 202502CD</th>
</tr>
</thead>
<tbody>
<tr>
<td>Date cast</td>
<td>08/05/09</td>
</tr>
<tr>
<td>Compression strength</td>
<td>4084 psi / 28.2 MPa (170 day on 01/23/2010)</td>
</tr>
<tr>
<td>Target crack opening (in.)</td>
<td>0.5 mm (0.0197&quot;)</td>
</tr>
<tr>
<td>Drill type</td>
<td>Hilti TE40-ARV</td>
</tr>
<tr>
<td>Drill bit Ø dcut (in.)</td>
<td>(1/2&quot; nominal)</td>
</tr>
<tr>
<td>Drill hole depth h1 (in.)</td>
<td>4.0&quot;</td>
</tr>
<tr>
<td>Effective depth hef (in.)</td>
<td>3.75&quot;</td>
</tr>
<tr>
<td>Drill hole cleaning</td>
<td>3xblow</td>
</tr>
<tr>
<td>Installation torque</td>
<td>40 ft-lb installation reduced to 20 ft-lb before test</td>
</tr>
<tr>
<td>Slab / NCS position</td>
<td>Slab 3, Position B</td>
</tr>
<tr>
<td>Data sample rate (Hz)</td>
<td>200 Hz</td>
</tr>
</tbody>
</table>

### Instrumentation

- **Crack Width Pots**: LP01, LP02, LP03, LP04
- **SAMU 1 (A-D)**: S18, S23, S3, S4
- **SAMU 2 (A-D)**: S5, S6, S7, S8
- **SAMU 3 (A-D)**: S9, S10, S11, S12
- **SAMU 4 (A-D)**: S13, S14, S15, S16
- **LW1 - LW4**: 229931, 239833, 239835, 239835
- **Disp N1-N4**: 119587, 119030, 119032, 122033
- **Disp V1-V2**: 122875, 122879
- **Disp Long, Disp Tran**: 25458, 0003-11971
- **Accl 1-8**: 119034, 119030, 1375, 1736, 1738
- **Rmax (in.)**: --
- **Rmin (in.)**: --
- **h (in.)**: --
- **Permanent displacement after test (in.)**: 1.39, 0.22, 0.21, 1.64

### Anchor projection

- Height above concrete was recorded "pre" and "post" application of installation torque

### Loading / Setup

- **SE (1)**, **NE (2)**, **SW (3)**, **NW (4)**
- **Drill hole depth h1 (in.)**: 3.967, 3.990, 3.991, 3.996
- **Height abv. conc. (in.) pre.**: 3.02, 3.02, 3.01, 3.02
- **Hairline crack mm (mm) pre/post**: 0.16 / 0.18, 0.15 / 0.20, 0.16 / 0.18, 0.14 / 0.18
- **Crack opening \(\Delta w\) (mm)**: 0.5, 0.5, 0.5, 0.5
- **Anchor preload (lbf)**: 50% Installation Torque (40 / 20 ft-lb)
- **Height abv. conc. (in.) post**: 3.28, 3.24, 3.25, 3.24
- **Anchor disp. during set (in.)**: 0.24, 0.22, 0.25, 0.22
- **Fauilure Mode**: Pt, Pt, Pt, Pt
- **Rmax (in.)**: --, --, --, --
- **Rmin (in.)**: --, --, --, --
- **h (in.)**: --, --, --, --
- **Permanent displacement after test (in.)**: 1.39, 0.22, 0.21, 1.64

### Key

- C = Concrete failure
- S = Steel failure
- Po = Pullout (including sleeve)
- Pt = Pull-through
- Sp = Splitting

---

### Test Date

1/27/2010

### Engineer(s)

Derrick Watkins, Arnold Gastelum
Allison Yu, Yujia Liu

### Technician(s)

Paul Grecco

---

Dynamic tests of the WALLE in cracked concrete.

**Concrete specimen**

Low-strength Vulcan 202502CD

**Date cast**

08/05/09

**Compression strength**

4084 psi / 28.2 MPa (170 day on 01/23/2010)

**Target crack opening (in.)**

0.5 mm (0.0197")

**Drill type**

Hilti TE40-ARV

**Drill bit Ø dcut (in.)**

(1/2" nominal)

**Drill hole depth h1 (in.)**

4.0"

**Effective depth hef (in.)**

3.75"

**Drill hole cleaning**

3xblow

**Installation torque**

40 ft-lb installation reduced to 20 ft-lb before test

**Slab / NCS position**

Slab 3, Position B

**Data sample rate (Hz)**

200 Hz

**Loading / Setup**

- **SE (1)**, **NE (2)**, **SW (3)**, **NW (4)**
- **Drill hole depth h1 (in.)**: 3.967, 3.990, 3.991, 3.996
- **Height abv. conc. (in.) pre.**: 3.02, 3.02, 3.01, 3.02
- **Hairline crack mm (mm) pre/post**: 0.16 / 0.18, 0.15 / 0.20, 0.16 / 0.18, 0.14 / 0.18
- **Crack opening \(\Delta w\) (mm)**: 0.5, 0.5, 0.5, 0.5
- **Anchor preload (lbf)**: 50% Installation Torque (40 / 20 ft-lb)
- **Height abv. conc. (in.) post**: 3.28, 3.24, 3.25, 3.24
- **Anchor disp. during set (in.)**: 0.24, 0.22, 0.25, 0.22
- **Fauilure Mode**: Pt, Pt, Pt, Pt
- **Rmax (in.)**: --, --, --, --
- **Rmin (in.)**: --, --, --, --
- **h (in.)**: --, --, --, --
- **Permanent displacement after test (in.)**: 1.39, 0.22, 0.21, 1.64

### Comments:

Anchor projection: Height above concrete was recorded "pre" and "post" application of installation torque
**Hilti Seismic Project - WALLE Dynamic Data Sheet**

**Series** V2_WALLE_F_N5_CC05_FM02  
**Date** 1/27/2010

PIA = Peak Input Acceleration  
PID = Peak Input Displacement

<table>
<thead>
<tr>
<th>Test name</th>
<th>Anchor</th>
<th>Type</th>
<th>Max. Crack Width</th>
<th>Table Loading</th>
<th>Crack Record</th>
<th>Motion</th>
<th>Bldg</th>
<th>Floor</th>
<th>Scale (%)</th>
<th>Scaled PIA (g)</th>
<th>Scaled PID (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>V2_WALLE_F_N5_CC05_FM02_10P</td>
<td>N5</td>
<td>Expansion</td>
<td>0.5mm</td>
<td>FM02</td>
<td>CR02_05</td>
<td>kobe00</td>
<td>2st</td>
<td>1</td>
<td>10</td>
<td>0.10</td>
<td>0.37</td>
</tr>
<tr>
<td>V2_WALLE_F_N5_CC05_FM02_30P</td>
<td>N5</td>
<td>Expansion</td>
<td>0.5mm</td>
<td>FM02</td>
<td>CR02_05</td>
<td>kobe00</td>
<td>2st</td>
<td>1</td>
<td>30</td>
<td>0.29</td>
<td>1.10</td>
</tr>
<tr>
<td>V2_WALLE_F_N5_CC05_FM02_100P</td>
<td>N5</td>
<td>Expansion</td>
<td>0.5mm</td>
<td>FM02</td>
<td>CR02_05</td>
<td>kobe00</td>
<td>2st</td>
<td>1</td>
<td>100</td>
<td>0.95</td>
<td>3.67</td>
</tr>
</tbody>
</table>

**TEST SETUP**

**DETAILED ANCHOR FAILURE PHOTOS**

- Anchor 4 (N-W)
- Anchor 2 (N-E)
- Anchor 3 (S-W)
- Anchor 1 (S-E)

**SIDE VIEW**

**PLAN VIEW FAILURE PHOTO**
Max Accel = 0.75 g

Abs. Accel.

Slab Accel

Max Displacement = 0.86 in

Abs. Disp.

Slab Disp

Max WALLE Accel = 1.21 g

Abs. Accel. @ WALLE CM

WALLE Accel

Max WALLE Disp = 0.79 in

Abs. Disp. @ WALLE CM

WALLE Disp

Time, sec

V2_WALLE_F_N5_CC05_FM02_30P

AnchorType: Expansion

Spectral Accel, g

WALLE Drift Ratio, %

WALLE Accel, g

2% Damping

Period, sec
V2_WALLE_F_N5_CC05_FM02_30P

Anchor Type: Expansion

Crack Width

Anchor Load

Anchor Disp

Time, sec

NorthWest
NorthEast
SouthWest
SouthEast

Monotonic Uncracked
Monotonic Cracked

Anchor Load

Crack Width

Crack Width

Anchor Load
Max Accel = 1.70 g

Abs. Accel.

Slab Accel

WALLE Accel g

Max WALLE Accel = 2.08 g

Abs. Accel. @ WALLE CM

WALLE Displacement

Max WALLE Disp = 6.16 in

Abs. Disp. @ WALLE CM

Abs. Disp.

Slab Displacement

Time, sec

Abs. Disp.

Max Disp = 2.89 in

0 2 4 6 8 10 12 14 16

Slab Accel

0 1 2 3 4 5 6 7

Slab Disp

0 5 10 15

WALLE Accel

0 1 2 3 4 5 6 7

WALLE Disp

0 1 2 3 4 5 6 7 8

Spectral Accel, g

0 1 2 3 4 5 6 7 8

WALLE Drift Ratio, %

0 1 2 3 4 5 6 7 8 9 10

WALLE Period

0.01 0.1 1 10

2% Damping

AnchorType: Expansion
Dynamic failure tests of the WALLE in cracked concrete.

**Test description**

Anchor & S/N: N5 - Expansion

Load / Wcr History: FM02 / CR02 (0.8mm max. crack width)

**Installation parameters**

Concrete specimen: Low-strength Vulcan 202502CD

Date cast: 08/05/09

Compression strength: 3798 psi / 26.2 MPa (64 day on 10/08/2009)

Target crack opening (in.): 0.8 mm (0.0315")

Drill type: Hilti TE40-ARV

Drill bit Ø dcut (in.): 0.5" (Recommended) 0.515" (Measured)

Drill hole depth h1 (in.): 4.125"

Effective depth hef (in.): 3.75"

Drill hole cleaning: 3xblow, 3xblow, 3xblow

Installation torque: 40 ft-lb, then release to 20 ft-lb

Slab / NCS position: Slab 4, Position A

Data sample rate (Hz): 200 Hz

**Instrumentation**

Crack Width Pots: LP01, LP02, LP03, LP04

SAMU 1 (A-D): S18, S2, S3, S4

SAMU 2 (A-D): S5, S6, S7, S8

SAMU 3 (A-D): S9, S10, S11, S12

SAMU 4 (A-D): S13, S14, S15, S16

Disp N1-N4: 119030, 119031, 119032, 122033

Disp V1-V2: 122875, 122879

Disp Long, Disp Tran: 20498, 2003-11971

LP01, LP02, LP03, LP04

**Anchor**

SE (1) | NE (2) | SW (3) | NW (4)

Drill hole depth h1 (in.): 4.18 | 4.17 | 4.17 | 4.15

Height above concrete (in.): 2.964 | 2.980 | 2.903 | 2.970

Hairline crack (mm): 0.04 | 0.08 | 0.10 | 0.12

Crack opening ∆w (mm): 0.8 | 0.8 | 0.8 | 0.8

Anchor preload: 20 ft-lb (~1900 lbs)

Failure load Fu (lbf): 0.176 | 0.380 | 0.208 | 0.144

Failure Mode: Pt | Pt | Pt | Pt

Rmax (in.): n/a | n/a | n/a | n/a

Rmin (in.): n/a | n/a | n/a | n/a

h (in.): n/a | n/a | n/a | n/a

**Loading / Setup**

Slab 4, Position A

Data sample rate (Hz): 200 Hz

Rod 1-4: R5, R2, R3, R4

**Comments:**

Key:

C = Concrete failure

S = Steel failure

Pa = Pullout (including sleeve)

Pt = Pull-through

Sp = Splitting
### TEST MOTION PARAMETERS

<table>
<thead>
<tr>
<th>Test name</th>
<th>Anchor</th>
<th>Type</th>
<th>Crack Width</th>
<th>Table Loading</th>
<th>Crack Record</th>
<th>Motion</th>
<th>Bldg</th>
<th>Floor</th>
<th>Scale Factor (%)</th>
<th>Scaled PIA (g)</th>
<th>Scaled PID (in)</th>
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</thead>
<tbody>
<tr>
<td>WALLE_F_N5_Hammer_2500lb</td>
<td>N5</td>
<td>Expansion</td>
<td>0.0mm</td>
<td>n/a</td>
<td>--</td>
<td>Hammer</td>
<td>--</td>
<td>--</td>
<td>--</td>
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<tr>
<td>WALLE_F_N5_Hammer_0lb</td>
<td>N5</td>
<td>Expansion</td>
<td>0.0mm</td>
<td>n/a</td>
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<td>Hammer</td>
<td>--</td>
<td>--</td>
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<td>--</td>
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<tr>
<td>WALLE_F_N5_UC_WIN</td>
<td>N5</td>
<td>Expansion</td>
<td>0.0mm</td>
<td>0.05WN CR02_08</td>
<td>WhiteNoise n/a</td>
<td>n/a</td>
<td>100</td>
<td>0.03</td>
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<td>WALLE_F_N5_CC08_WN</td>
<td>N5</td>
<td>Expansion</td>
<td>0.8mm</td>
<td>0.05WN CR02_08</td>
<td>WhiteNoise n/a</td>
<td>n/a</td>
<td>100</td>
<td>0.03</td>
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<tr>
<td>WALLE_F_N5_CC08_FM02_10P</td>
<td>N5</td>
<td>Expansion</td>
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<td>FM02 CR02_08</td>
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<td>10</td>
<td>0.10</td>
<td>0.37</td>
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<td>Expansion</td>
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<td>FM02 CR02_08</td>
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<td>0.29</td>
<td>1.10</td>
<td>--</td>
<td>--</td>
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<tr>
<td>WALLE_F_N5_CC08_FM02_100P</td>
<td>N5</td>
<td>Expansion</td>
<td>0.8mm</td>
<td>FM02 CR02_08</td>
<td>kobe00</td>
<td>2st</td>
<td>100</td>
<td>0.95</td>
<td>3.67</td>
<td>--</td>
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<td>WALLE_F_N5_CC08_FM02_125P</td>
<td>N5</td>
<td>Expansion</td>
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<td>FM02 CR02_08</td>
<td>kobe00</td>
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<td>125</td>
<td>1.19</td>
<td>4.69</td>
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</tr>
</tbody>
</table>

### TEST SETUP

- **Anchor 4 (N-W)**
- **Anchor 2 (N-E)**
- **Anchor 3 (S-W)**
- **Anchor 1 (S-E)**

### DETAILED ANCHOR FAILURE PHOTOS

- **SIDE VIEW**
- **FAILURE PHOTO**

Note: Plan view photo not available.

All anchors had Pultthrough failure.
Test was conducted at SDSU as part of uncracked concrete test program, data included for comparison with cracked concrete case.
Test was conducted at SDSU as part of uncracked concrete test program, data included for comparison with cracked concrete case.
Test was conducted at SDSU as part of uncracked concrete test program, data included for comparison with cracked concrete case.
Test was conducted at SDSU as part of uncracked concrete test program, data included for comparison with cracked concrete case.
Max Accel = 1.56 g
Max Disp = 1.34 in
Max WALLE Accel = 2.01 g
Max WALLE Disp = 2.44 in

AnchorType: Expansion
Crack Width

Anchor Load

Anchor Disp

Time, sec

WALLE_F_N5_CC08_FM02_100P

Anchor Type: Expansion

Max NW = 0.71 mm
Max NE = 0.74 mm
Max SW = 1.10 mm
Max SE = 1.12 mm

Max NW = 5.87 kip
Max NE = 8.10 kip
Max SW = 5.23 kip
Max SE = 4.86 kip

Max NW = 0.26 in
Max NE = 0.23 in
Max SW = 0.54 in
Max SE = 0.56 in

Crack Width

Anchor Load

Anchor Disp

Time, sec
## Test description
Dynamic tests of the WALLE in cracked concrete.

## Anchor & S/N
N9 - Undercut
(modified embedment depth)

## Load / Wcr History
FM02 / CR02 (0.8mm max. crack width)

### Installation parameters

<table>
<thead>
<tr>
<th>Concrete specimen</th>
<th>Low-strength Vulcan 202502CD</th>
</tr>
</thead>
<tbody>
<tr>
<td>Date cast</td>
<td>06/05/09</td>
</tr>
<tr>
<td>Compression strength</td>
<td>3798 psi / 26.2 MPa (64 day on 10/08/2009)</td>
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<tr>
<td>Target crack opening (in.)</td>
<td>0.8 mm (0.0315&quot;)</td>
</tr>
<tr>
<td>Drill type</td>
<td>Hilti TE40-ARV</td>
</tr>
<tr>
<td>Drill bit Ø dcut (in.)</td>
<td>0.787&quot;, 0.8012&quot; measured (20mm, 20.35mm measured)</td>
</tr>
<tr>
<td>Drill hole depth h1 (in.)</td>
<td>2.625&quot;</td>
</tr>
<tr>
<td>Effective depth hef (in.)</td>
<td>2.4&quot;</td>
</tr>
<tr>
<td>Drill hole cleaning</td>
<td>3xblow</td>
</tr>
<tr>
<td>Installation torque</td>
<td>None, Set w/ tool &amp; drill on hammer &amp; drill</td>
</tr>
<tr>
<td>Slab / NCS position</td>
<td>Slab 4, Position B, Top Side</td>
</tr>
<tr>
<td>Data sample rate (Hz)</td>
<td>200 Hz</td>
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### Instrumentation

<table>
<thead>
<tr>
<th>Crack Width Pots</th>
<th>SAMU 1 (A-D)</th>
<th>S18, S2, S3, S4</th>
</tr>
</thead>
<tbody>
<tr>
<td>SAMU 2 (A-D)</td>
<td>S5, S6, S7, S8</td>
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</tr>
<tr>
<td>SAMU 3 (A-D)</td>
<td>S9, S10, S11, S12</td>
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<tr>
<td>SAMU 4 (A-D)</td>
<td>S13, S14, S15, S16</td>
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</table>

<table>
<thead>
<tr>
<th>LW1 - LW4</th>
<th>229931, 239833, 239834, 239835</th>
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</thead>
<tbody>
<tr>
<td>119587, 119030, 119032, 122033</td>
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<tr>
<td>Disp N1-N4</td>
<td>122875, 122879</td>
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<tr>
<td>Disp V1-V2</td>
<td>20498, 0003-11971</td>
</tr>
<tr>
<td>Disp Long, Disp Tran</td>
<td>20498, 0003-11971</td>
</tr>
<tr>
<td>Slab 4, Position B, Top Side</td>
<td>Accl 1-8</td>
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<tr>
<td>860, 1386, 1733, 1734, 1735, 1736, 1737, 1738</td>
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<tr>
<td>Rod 1-4</td>
<td>R5, R2, R3, R4</td>
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</table>

### Loading / Setup

<table>
<thead>
<tr>
<th>Anchor</th>
<th>SE (1)</th>
<th>NE (2)</th>
<th>SW (3)</th>
<th>NW (4)</th>
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</thead>
<tbody>
<tr>
<td>Drill hole depth h1 (in.)</td>
<td>2.73</td>
<td>2.74</td>
<td>2.73</td>
<td>2.73</td>
</tr>
<tr>
<td>Height above concrete (in.)</td>
<td>3.05</td>
<td>3.09</td>
<td>3.12</td>
<td>3.11</td>
</tr>
<tr>
<td>Hairline crack (mm)</td>
<td>0.06</td>
<td>0.04</td>
<td>0.08</td>
<td>0.12</td>
</tr>
<tr>
<td>Crack opening ∆w (mm)</td>
<td>0.8</td>
<td>0.8</td>
<td>0.8</td>
<td>0.8</td>
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</table>

<table>
<thead>
<tr>
<th>Anchor preload (lbf)</th>
<th>50 lbs</th>
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</thead>
<tbody>
<tr>
<td>Anchor disp. during set (in)</td>
<td>0.33</td>
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<tr>
<td>Failure Mode</td>
<td>C</td>
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<tr>
<td>Rmax (in.)</td>
<td>11</td>
</tr>
<tr>
<td>Rmin (in.)</td>
<td>8</td>
</tr>
<tr>
<td>h (in.)</td>
<td>2.25</td>
</tr>
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</table>

### Comments:

- **Key**
  - C = Concrete failure
  - S = Steel failure
  - Po = Pullout (including sleeve)
  - Pt = Pull-through
  - Sp = Splitting
## Test Motion Parameters

<table>
<thead>
<tr>
<th>Test name</th>
<th>Anchor</th>
<th>Type</th>
<th>Crack Width</th>
<th>Table Loading</th>
<th>Crack Record</th>
<th>Motion</th>
<th>Bldg</th>
<th>Floor</th>
<th>Target Scale (%)</th>
<th>Scaled PIA (g)</th>
<th>Scaled PID (in)</th>
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</thead>
<tbody>
<tr>
<td>WALLE_F_N7_CC08_FM02_5P</td>
<td>N7</td>
<td>Drop-In</td>
<td>0.8mm</td>
<td>FM02</td>
<td>CR02_08</td>
<td>kobe00</td>
<td>2st</td>
<td>1</td>
<td>5</td>
<td>0.05</td>
<td>0.18</td>
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<tr>
<td>WALLE_F_N7_CC08_FM02_15P</td>
<td>N7</td>
<td>Drop-In</td>
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<td>2st</td>
<td>1</td>
<td>15</td>
<td>0.14</td>
<td>0.55</td>
</tr>
<tr>
<td>WALLE_F_N7_CC08_FM02_20P</td>
<td>N7</td>
<td>Drop-In</td>
<td>0.8mm</td>
<td>FM02</td>
<td>CR02_08</td>
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<td>2st</td>
<td>1</td>
<td>20</td>
<td>0.19</td>
<td>0.73</td>
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<tr>
<td>WALLE_F_N7_CC08_FM02_25P</td>
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<td>Drop-In</td>
<td>0.8mm</td>
<td>FM02</td>
<td>CR02_08</td>
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<td>2st</td>
<td>1</td>
<td>25</td>
<td>0.24</td>
<td>0.92</td>
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<td>WALLE_F_N7_CC08_FM02_35P</td>
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<td>Drop-In</td>
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<td>CR02_08</td>
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<td>2st</td>
<td>1</td>
<td>35</td>
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<td>1.29</td>
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<tr>
<td>WALLE_F_N7_CC08_FM02_60P</td>
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<td>Drop-In</td>
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<td>CR02_08</td>
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<td>1</td>
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<tr>
<td>WALLE_F_N7_CC08_FM02_100P</td>
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<td>2st</td>
<td>1</td>
<td>100</td>
<td>0.95</td>
<td>3.67</td>
</tr>
</tbody>
</table>

## Test Setup (looking South)

![Anchor 4 (N-W)](image1)

![Anchor 2 (N-E)](image2)

![Anchor 3 (S-W)](image3)

![Anchor 1 (S-E)](image4)

## Detailed Anchor Failure Photos

![Anchor 4 (N-W)](image5)

![Anchor 2 (N-E)](image6)

![Anchor 3 (S-W)](image7)

![Anchor 1 (S-E)](image8)

## Side View (looking West)

![Side View](image9)

## Plan View Failure Photo

![Plan View](image10)
WALLE_F_N7_CC08_FM02_60P

AnchorType: Drop-In

Crack Width

<table>
<thead>
<tr>
<th></th>
<th>Max NW</th>
<th>Max NE</th>
<th>Max SW</th>
<th>Max SE</th>
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<tbody>
<tr>
<td>NW</td>
<td>0.71 mm</td>
<td>0.72 mm</td>
<td>0.85 mm</td>
<td>0.73 mm</td>
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<tr>
<td>NE</td>
<td>2.33 kip</td>
<td>4.85 kip</td>
<td>4.81 kip</td>
<td>3.80 kip</td>
</tr>
<tr>
<td>SW</td>
<td></td>
<td></td>
<td>0.35 in</td>
<td>0.37 in</td>
</tr>
<tr>
<td>SE</td>
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Anchor Load

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<th>Max NE</th>
<th>Max SW</th>
<th>Max SE</th>
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<tbody>
<tr>
<td>NW</td>
<td>2.33 kip</td>
<td>4.85 kip</td>
<td>4.81 kip</td>
<td>3.80 kip</td>
</tr>
<tr>
<td>NE</td>
<td>0.18 in</td>
<td>0.18 in</td>
<td>0.35 in</td>
<td>0.37 in</td>
</tr>
<tr>
<td>SW</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SE</td>
<td></td>
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Time, sec

Anchor Disp

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<th>Max NW</th>
<th>Max NE</th>
<th>Max SW</th>
<th>Max SE</th>
</tr>
</thead>
<tbody>
<tr>
<td>NW</td>
<td>0.18 in</td>
<td>0.18 in</td>
<td>0.35 in</td>
<td>0.37 in</td>
</tr>
<tr>
<td>NE</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SW</td>
<td></td>
<td></td>
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</tr>
<tr>
<td>SE</td>
<td></td>
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</tbody>
</table>
**Hilt Seismic Project - WALLE Dynamic Data Sheet**

**Series**
WALLE_F_N9_CC08_FM02

**Test Date**
11/02/09

**Engineer(s)**
Derrick Watkins, Arnold Gastelum
Allison Yu, Yujia Liu

**Technician(s)**
Paul Grecco

---

**Test description**
Dynamic tests of the WALLE in cracked concrete.

**Anchor & S/N**
N9 - Undercut
(modified embedment depth)

**Load / Wcr History**
FM02 / CR02 (0.8mm max. crack width)

---

### Installation parameters | Instrumentation

| Concrete specimen | Low-strength Vulcan 202502CD | Table, Act E&W P&A |
| Date cast | 08/05/09 | Crack Width Pots |
| Compression strength | 3798 psi / 26.2 MPa (64 day on 10/08/2009) | SAMU 1 (A-D) |
| Target crack opening (in.) | 0.8 mm (0.0315") | SAMU 2 (A-D) |
| Drill type | Hilti TE40-ARV | SAMU 3 (A-D) |
| Drill bit Ø dcut (in.) | 0.787", 0.8012" measured (20mm, 20.35mm measured) | SAMU 4 (A-D) |
| Drill hole depth h1 (in.) | 2.625" | LW1 - LW4 |
| Effective depth hef (in.) | 2.625" | 229931, 239833, 239835 |
| Drill hole cleaning | 3 x blow | 229931, 239833 |
| Installation torque | N/A | 239835 |
| Slab / NCS position | Slab 4, Position C, Bottom Side | LP01, LP02, LP03, LP04 |
| Slab sample rate (Hz) | 200 Hz | LP04 |

**Anchor**

<table>
<thead>
<tr>
<th></th>
<th>SE (1)</th>
<th>NE (2)</th>
<th>SW (3)</th>
<th>NW (4)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Height above concrete (in.)</td>
<td>3.10</td>
<td>3.05</td>
<td>3.10</td>
<td>3.14</td>
</tr>
<tr>
<td>Hairline crack (mm)</td>
<td>0.04</td>
<td>0.04</td>
<td>0.04</td>
<td>0.04</td>
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<tr>
<td>Crack opening ∆w (mm)</td>
<td>0.8</td>
<td>0.8</td>
<td>0.8</td>
<td>0.8</td>
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<tr>
<td>Anchor preload (lb)</td>
<td>50 lbs</td>
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**Faulture Mode**

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<th>C</th>
<th>n/a</th>
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<tbody>
<tr>
<td>Rmax (in.)</td>
<td>11</td>
<td>n/a</td>
<td>10</td>
<td>n/a</td>
</tr>
<tr>
<td>Rmin (in.)</td>
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<td>n/a</td>
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<tr>
<td>h (in.)</td>
<td>2.25</td>
<td>n/a</td>
<td>2.56</td>
<td>n/a</td>
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</tbody>
</table>

**Comments:**

- **Key**
  - C = Concrete failure
  - S = Steel failure
  - Po = Pullout (including sleeve)
  - Pt = Pull-through
  - Sp = Splitting
## TEST MOTION PARAMETERS

<table>
<thead>
<tr>
<th>Test name</th>
<th>Anchor</th>
<th>Type</th>
<th>Crack Width</th>
<th>Table Loading</th>
<th>Crack Record</th>
<th>Motion</th>
<th>Scale Factor (%)</th>
<th>Scaled PIA (g)</th>
<th>Scaled PID (in)</th>
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</thead>
<tbody>
<tr>
<td>WALLE_F_N9_CC08_FM02_15P</td>
<td>N9</td>
<td>Undercut</td>
<td>0.8mm</td>
<td>FM02</td>
<td>CR02_08</td>
<td>kobe00</td>
<td>1</td>
<td>15</td>
<td>0.14</td>
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<td>WALLE_F_N9_CC08_FM02_20P</td>
<td>N9</td>
<td>Undercut</td>
<td>0.8mm</td>
<td>FM02</td>
<td>CR02_08</td>
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<td>0.19</td>
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<td>WALLE_F_N9_CC08_FM02_40P</td>
<td>N9</td>
<td>Undercut</td>
<td>0.8mm</td>
<td>FM02</td>
<td>CR02_08</td>
<td>kobe00</td>
<td>1</td>
<td>40</td>
<td>0.38</td>
</tr>
<tr>
<td>WALLE_F_N9_CC08_FM02_60P</td>
<td>N9</td>
<td>Undercut</td>
<td>0.8mm</td>
<td>FM02</td>
<td>CR02_08</td>
<td>kobe00</td>
<td>1</td>
<td>60</td>
<td>0.57</td>
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</tbody>
</table>

## TEST SETUP

### DETAILED ANCHOR FAILURE PHOTOS

- Anchor 4 (N-W)
- Anchor 2 (N-E)
- Anchor 3 (S-W)
- Anchor 1 (S-E)

### SIDE VIEW (not available)

### PLAN VIEW FAILURE PHOTO
Max Accel = 1.11 g
Abs. Accel.

Max Disp = 1.15 in
Abs. Disp.

Max WALLE Accel = 1.60 g
Abs. Accel. @ WALLE CM

Max WALLE Disp = 1.06 in
Abs. Disp. @ WALLE CM

WALLE Drift Ratio, %

2% Damping

Period, sec

WALLE Accel, g

Spectral Accel, g

WALLE Accel, g

WALLE Drift Ratio, %

Period, sec
WALLE_F_N9_CC08_FM02_60P

AnchorType: Undercut

Max Accel = 1.31 g
Max WALLE Accel = 1.92 g

Max Disp = 1.73 in
Max WALLE Disp = 3.85 in

2% Damping
WALLE_F_N9_CC08_FM02_60P

Anchor Type: Undercut

Crack Width vs Time, sec

- Max NW = 0.67 mm
- Max NE = 0.66 mm
- Max SW = 1.05 mm
- Max SE = 1.08 mm

Anchor Load vs Time, sec

- Max NW = 9.13 kip
- Max NE = 10.85 kip
- Max SW = 7.82 kip
- Max SE = 6.59 kip

Anchor Disp vs Time, sec

- Max NW = 0.11 in
- Max NE = 0.13 in
- Max SW = 1.02 in
- Max SE = 1.04 in

Crack Width vs Anchor Load

- Monotonic Uncracked
- Monotonic Cracked

Crack Width vs Anchor Disp

- NW, NE, SW, SE
Hilti Seismic Project - WALLE Dynamic Data Sheet

**Test description**
Dynamic tests of the WALLE in cracked concrete.

**Anchor & S/N**
N9 - Undercut (modified embedment depth)

**Load / Wcr History**
FM02 / CR02 (0.8mm max. crack width)

### Installation parameters

<table>
<thead>
<tr>
<th>Concrete specimen</th>
<th>Low-strength Vulcan 202502CD</th>
</tr>
</thead>
<tbody>
<tr>
<td>Date cast</td>
<td>08/05/09</td>
</tr>
<tr>
<td>Compression strength</td>
<td>3798 psi / 26.2 MPa (64 days on 10/08/2009)</td>
</tr>
<tr>
<td>Target crack opening (in.)</td>
<td>0.8 mm (0.031&quot;&quot;)</td>
</tr>
<tr>
<td>Drill type</td>
<td>Hilti TE40-ARV</td>
</tr>
<tr>
<td>Drill bit Ø dcut (in.)</td>
<td>0.787&quot;, 0.8012&quot; measured (20mm, 20.30mm measured)</td>
</tr>
<tr>
<td>Drill hole depth h1 (in.)</td>
<td>2.625&quot;</td>
</tr>
<tr>
<td>Effective depth hcf (in.)</td>
<td>2.4&quot;</td>
</tr>
<tr>
<td>Drill hole cleaning</td>
<td>3x blow</td>
</tr>
<tr>
<td>Installation torque</td>
<td>none, Set with tool &amp; drill on hammer &amp; drill</td>
</tr>
<tr>
<td>Slab / NCS position</td>
<td>Slab 4, Position B, Top Side</td>
</tr>
<tr>
<td>Data sample rate (Hz)</td>
<td>200 Hz</td>
</tr>
</tbody>
</table>

### Instrumentation

| Crack Width Pots | LP01, LP02, LP03, LP04 |
| SAMU 1 (A-D) | S18, S2, S3, S4 |
| SAMU 2 (A-D) | S5, S6, S7, S8 |
| SAMU 3 (A-D) | S9, S10, S11, S12 |
| SAMU 4 (A-D) | S13, S14, S15, S16 |

### Anchor

<table>
<thead>
<tr>
<th>SE (1)</th>
<th>NE (2)</th>
<th>SW (3)</th>
<th>NW (4)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.73</td>
<td>2.74</td>
<td>2.73</td>
<td>2.72</td>
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</table>

### Loading / Setup

<table>
<thead>
<tr>
<th>L = 96&quot;</th>
<th>W = 42&quot;</th>
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</thead>
<tbody>
<tr>
<td>S1 = 24&quot;</td>
<td>S2 = 18&quot;</td>
</tr>
<tr>
<td>C1 = 3[0]^&quot;</td>
<td>C2 = 3[0]^&quot;</td>
</tr>
</tbody>
</table>

### Anchor preload (lbf)

<table>
<thead>
<tr>
<th>50 lbs</th>
<th>50 lbs</th>
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</thead>
</table>

### Failure Mode

<table>
<thead>
<tr>
<th>C = Concrete failure</th>
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</thead>
</table>

### Crack opening \(\Delta w\) (mm)

<table>
<thead>
<tr>
<th>0.8</th>
<th>0.8</th>
</tr>
</thead>
</table>

### Installation torque

<table>
<thead>
<tr>
<th>Disp V1-V2</th>
<th>Disp V2-V4</th>
</tr>
</thead>
<tbody>
<tr>
<td>122875, 122879</td>
<td>0003-11971</td>
</tr>
</tbody>
</table>

### Slab / NCS position

<table>
<thead>
<tr>
<th>Slab 4, Position B, Top Side</th>
</tr>
</thead>
</table>

### Data sample rate (Hz)

<table>
<thead>
<tr>
<th>200 Hz</th>
</tr>
</thead>
</table>

### Comments:

- Key:
  - C = Concrete failure
  - S = Steel failure
  - Po = Pullout (Including sleeve)
  - Pt = Pull-through
  - Sp = Splitting

---

**Concrete specimen**

- Low-strength Vulcan 202502CD

**Date cast**

- 08/05/09

**Compression strength**

- 3798 psi / 26.2 MPa (64 days on 10/08/2009)

**Target crack opening (in.)**

- 0.8 mm (0.031"")

**Drill type**

- Hilti TE40-ARV

**Drill bit Ø dcut (in.)**

- 0.787", 0.8012" measured (20mm, 20.30mm measured)

**Drill hole depth h1 (in.)**

- 2.625"

**Effective depth hcf (in.)**

- 2.4"

**Drill hole cleaning**

- 3x blow

**Installation torque**

- none, Set with tool & drill on hammer & drill

**Slab / NCS position**

- Slab 4, Position B, Top Side

**Data sample rate (Hz)**

- 200 Hz

**Anchor preload (lbf)**

- 50 lbs

**Anchor disp. during set (in.)**

- 0.33

**Failure Mode**

- C = Concrete failure

**Rmax (in.)**

- 11

**Rmin (in.)**

- 8

**h (in.)**

- 2.25

---

**Concrete specimen**

- Low-strength Vulcan 202502CD

**Date cast**

- 08/05/09

**Compression strength**

- 3798 psi / 26.2 MPa (64 days on 10/08/2009)

**Target crack opening (in.)**

- 0.8 mm (0.031"")

**Drill type**

- Hilti TE40-ARV

**Drill bit Ø dcut (in.)**

- 0.787", 0.8012" measured (20mm, 20.30mm measured)

**Drill hole depth h1 (in.)**

- 2.625"

**Effective depth hcf (in.)**

- 2.4"

**Drill hole cleaning**

- 3x blow

**Installation torque**

- none, Set with tool & drill on hammer & drill

**Slab / NCS position**

- Slab 4, Position B, Top Side

**Data sample rate (Hz)**

- 200 Hz

**Anchor preload (lbf)**

- 50 lbs

**Anchor disp. during set (in.)**

- 0.33

**Failure Mode**

- C = Concrete failure

**Rmax (in.)**

- 11

**Rmin (in.)**

- 8

**h (in.)**

- 2.25

---

**Concrete specimen**

- Low-strength Vulcan 202502CD

**Date cast**

- 08/05/09

**Compression strength**

- 3798 psi / 26.2 MPa (64 days on 10/08/2009)

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**Drill type**

- Hilti TE40-ARV

**Drill bit Ø dcut (in.)**

- 0.787", 0.8012" measured (20mm, 20.30mm measured)

**Drill hole depth h1 (in.)**

- 2.625"

**Effective depth hcf (in.)**

- 2.4"

**Drill hole cleaning**

- 3x blow

**Installation torque**

- none, Set with tool & drill on hammer & drill

**Slab / NCS position**

- Slab 4, Position B, Top Side

**Data sample rate (Hz)**

- 200 Hz

**Anchor preload (lbf)**

- 50 lbs

**Anchor disp. during set (in.)**

- 0.33

**Failure Mode**

- C = Concrete failure

**Rmax (in.)**

- 11

**Rmin (in.)**

- 8

**h (in.)**

- 2.25
Hilti Seismic Project - WALLE Dynamic Data Sheet

Series: V2_WALLE_F_N9_CC08_Undercut
Date: 12/7/2009

PIA = Peak Input Acceleration
PID = Peak Input Displacement

## TEST MOTION PARAMETERS

<table>
<thead>
<tr>
<th>Test name</th>
<th>Anchor</th>
<th>Type</th>
<th>Crack Width</th>
<th>Table Loading</th>
<th>Crack Record</th>
<th>Motion</th>
<th>Bldg</th>
<th>Floor</th>
<th>Scale Factor (%)</th>
<th>Scaled PIA (g)</th>
<th>Scaled PID (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>V2_WALLE_F_N9_DC_WN3</td>
<td>N9</td>
<td>DC</td>
<td>0.0mm</td>
<td>0.03WN</td>
<td>n/a</td>
<td>WhiteNoise</td>
<td>n/a</td>
<td>n/a</td>
<td>100</td>
<td>0.03</td>
<td></td>
</tr>
<tr>
<td>V2_WALLE_F_N9_CC08_WN3</td>
<td>N9</td>
<td>CC</td>
<td>0.8mm</td>
<td>0.03WN</td>
<td>static 0.8</td>
<td>WhiteNoise</td>
<td>n/a</td>
<td>n/a</td>
<td>100</td>
<td>0.03</td>
<td></td>
</tr>
<tr>
<td>V2_WALLE_F_N9_CC08_FM02_20P</td>
<td>N9</td>
<td>CC</td>
<td>0.8mm</td>
<td>FM02</td>
<td>CR02_08</td>
<td>kobe00</td>
<td>2st</td>
<td>1</td>
<td>20</td>
<td>0.19</td>
<td>0.73</td>
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<tr>
<td>V2_WALLE_F_N9_CC08_FM02_40P</td>
<td>N9</td>
<td>CC</td>
<td>0.8mm</td>
<td>FM02</td>
<td>CR02_08</td>
<td>kobe00</td>
<td>2st</td>
<td>1</td>
<td>40</td>
<td>0.38</td>
<td>1.47</td>
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<tr>
<td>V2_WALLE_F_N9_CC08_FM02_60P</td>
<td>N9</td>
<td>CC</td>
<td>0.8mm</td>
<td>FM02</td>
<td>CR02_08</td>
<td>kobe00</td>
<td>2st</td>
<td>1</td>
<td>60</td>
<td>0.57</td>
<td>2.20</td>
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</tbody>
</table>

## TEST SETUP

- Anchor 4 (N-W)
- Anchor 2 (N-E)
- Anchor 3 (S-W)
- Anchor 1 (S-E)

## DETAILED ANCHOR FAILURE PHOTOS

- PLAN VIEW FAILURE PHOTO
- SIDE VIEW

- Anchor 4 (N-W)
- Anchor 2 (N-E)
- Anchor 3 (S-W)
- Anchor 1 (S-E)
V2_WALLE_F_N9_CC08_FM02_40P

AnchorType: Undercut

Crack Width

Anchor Disp

Anchor Load

Monotonic Uncracked

Monotonic Cracked

Crack Width

Anchor Load

Anchor Disp

Monotonic Uncracked

Monotonic Cracked

Crack Width

Anchor Load

Anchor Disp

Monotonic Uncracked

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Crack Width

Anchor Load

Anchor Disp

Monotonic Uncracked

Monotonic Cracked

Crack Width
Max Accel = 1.03 g
Abs. Accel.
Slab Accel

Max Disp = 1.73 in
Abs. Disp.
Slab Disp

Max WALLE Accel = 1.68 g
Abs. Accel. @ WALLE CM
WALLE Accel

Max WALLE Disp = 1.51 in
Abs. Disp. @ WALLE CM
WALLE Disp

V2_WALLE_F_N9_CC08_FM02_60P
AnchorType: Undercut
**Hilti Seismic Project - WALLE Dynamic Data Sheet**

**Series**
- STRUCT_N9_Undercut

**Date**
- 12/07/09

**Engineer(s)**
- Derrick Watkins, Arnold Gastelum
- Allison Yu, Yujia Liu

**Technician(e)**
- Paul Grecco

---

**Test description**
Dynamic tests of the WALLE in cracked concrete.

**Anchor & S/N**
- N9 - Undercut (modified embedment depth)

**Load / Wcr History**
- CP1 = Crack protocol (CP1)

**FM14 = Crack history**

---

### Installation parameters

<table>
<thead>
<tr>
<th>Concrete specimen</th>
<th>Low-strength Vulcan 202502CD</th>
</tr>
</thead>
<tbody>
<tr>
<td>Date cast</td>
<td>08/05/09</td>
</tr>
<tr>
<td>Compression strength</td>
<td>3798 psi / 26.2 MPa (84 day on 10/08/2009)</td>
</tr>
<tr>
<td>Target crack opening (in.)</td>
<td>0.8 mm (0.0315&quot;)</td>
</tr>
<tr>
<td>Drill type</td>
<td>Hilti TE40-ARV</td>
</tr>
<tr>
<td>Drill bit Ø dcut (in.)</td>
<td>0.787&quot;, 0.8012&quot; measured (20mm, 20.35mm measured)</td>
</tr>
<tr>
<td>Drill hole depth h1 (in.)</td>
<td>2.625&quot;</td>
</tr>
<tr>
<td>Effective depth hef (in.)</td>
<td>2.4&quot;</td>
</tr>
<tr>
<td>Drill hole cleaning</td>
<td>3x blow</td>
</tr>
<tr>
<td>Installation torque</td>
<td>none, Set w/ tool &amp; drill on hammer &amp; drill</td>
</tr>
<tr>
<td>Slab / NCS position</td>
<td>Slab 3, Position C, Side 2</td>
</tr>
<tr>
<td>Data sample rate (Hz)</td>
<td>200 Hz</td>
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### Instrumentation

<table>
<thead>
<tr>
<th>Crack Pots</th>
<th>SAMU 1 (A-D)</th>
<th>SAMU 2 (A-D)</th>
<th>SAMU 3 (A-D)</th>
<th>SAMU 4 (A-D)</th>
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<tbody>
<tr>
<td>LP01, LP02, LP03, LP04</td>
<td>S1, S2, S3, S4</td>
<td>S5, S6, S7, S8</td>
<td>S9, S10, S11, S12</td>
<td>S13, S14, S15, S16</td>
</tr>
</tbody>
</table>

### Anchor

<table>
<thead>
<tr>
<th>Anchor</th>
<th>Loading / Setup</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td>SE (1)</td>
<td>NE (2)</td>
</tr>
<tr>
<td>2.70</td>
<td>2.77</td>
</tr>
<tr>
<td>Height above concrete (in.)</td>
<td></td>
</tr>
<tr>
<td>Hairline crack (mm)</td>
<td></td>
</tr>
<tr>
<td>Crack opening ∆w (mm)</td>
<td>0.8</td>
</tr>
<tr>
<td>Anchor preload (lbf)</td>
<td>(not used)</td>
</tr>
<tr>
<td>Anchor disp. during set (in)</td>
<td></td>
</tr>
<tr>
<td>Failure Mode</td>
<td>C</td>
</tr>
<tr>
<td>Rmax (in.)</td>
<td>16.5</td>
</tr>
<tr>
<td>Rmin (in.)</td>
<td>13</td>
</tr>
<tr>
<td>h (in.)</td>
<td>2.1</td>
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</tbody>
</table>

### Comments:

- Key:
  - C = Concrete failure
  - S = Steel failure
  - Po = Pullout (including sleeve)
  - Pt = Pull-through
  - Sp = Splitting
### TEST MOTION PARAMETERS

<table>
<thead>
<tr>
<th>Test name</th>
<th>Anchor Type</th>
<th>Crack Record</th>
<th>Motion</th>
<th>Crack Width</th>
<th>Crack History Details</th>
<th>WALLE Displacement History</th>
</tr>
</thead>
<tbody>
<tr>
<td>STRUCT_N9_CC08_CP2_D1a</td>
<td>N9</td>
<td>Undercut CP 2Hz</td>
<td>CP 1Hz</td>
<td>0.8mm</td>
<td>Crack 0.8mm Disp 1Hz</td>
<td>Act 3, Disp=0.1&quot;, 1Hz</td>
</tr>
<tr>
<td>STRUCT_N9_CC08_CP2_D1b</td>
<td>N9</td>
<td>Undercut CP 2Hz</td>
<td>CP 1Hz</td>
<td>0.8mm</td>
<td>Crack 0.8mm Disp 1Hz</td>
<td>Act 3, Disp=0.25&quot;, 1Hz</td>
</tr>
<tr>
<td>STRUCT_N9_CC08_CP2_D1c</td>
<td>N9</td>
<td>Undercut CP 2Hz</td>
<td>CP 1Hz</td>
<td>0.8mm</td>
<td>Crack 0.8mm Disp 1Hz</td>
<td>Act 3, Disp=0.25&quot;, 1Hz</td>
</tr>
<tr>
<td>STRUCT_N9_CC08_CR14_D14a</td>
<td>N9</td>
<td>Undercut CR14</td>
<td>FM14</td>
<td>0.8mm</td>
<td>Crack 0.8mm FM14</td>
<td>Act 3, Disp=0.1&quot;</td>
</tr>
<tr>
<td>STRUCT_N9_CC08_CR14_D14b</td>
<td>N9</td>
<td>Undercut CR14</td>
<td>FM14</td>
<td>0.8mm</td>
<td>Crack 0.8mm FM14</td>
<td>Act 3, Disp=0.25&quot;</td>
</tr>
<tr>
<td>STRUCT_N9_CC08_CR14_D14c</td>
<td>N9</td>
<td>Undercut CR14</td>
<td>FM14</td>
<td>0.8mm</td>
<td>Crack 0.8mm FM14</td>
<td>Act 3, Disp=0.75&quot;</td>
</tr>
</tbody>
</table>

### TEST SETUP

- Anchor 4 (N-W)
- Anchor 2 (N-E)
- Anchor 3 (S-W)
- Anchor 1 (S-E)

### DETAILED ANCHOR FAILURE PHOTOS

![Anchor 4 (N-W)](image)

![Anchor 2 (N-E)](image)

![Anchor 3 (S-W)](image)

![Anchor 1 (S-E)](image)

### SIDE VIEW

![Side View](image)

### PLAN VIEW FAILURE PHOTO

![Plan View](image)
STRUCT_N9_CC08_CP2_D1b

AnchorType: Undercut

Max Accel = 0.59 g

Abs. Accel.

Slab Accel

g

Max Disp = 0.01 in

Abs. Disp.

Slab Disp

in

Max WALLE Accel = 0.92 g

Abs. Accel. @ WALLE CM

WALLE Accel

g

Max WALLE Disp = 0.97 in

Abs. Disp. @ WALLE CM

WALLE Disp

in

Time, sec

0 5 10 15 20 25 30 35 40 45

2% Damping

Spectral Accel, g

Period, sec

0.01 0.1 1 10

WALLE Drift Ratio, %

WALLE Accel, g

2.5 2 1.5 1 0.5 0

-1 0 1

0 5 10 15 20 25 30 35 40 45
STRUCT_N9_CC08_CP2_D1b

Anchor Type: Undercut

![Graph showing crack width, anchor load, and anchor displacement over time.](image)

- Crack Width:
  - NorthWest (NW): Max NW = 0.86 mm
  - NorthEast (NE): Max NE = 0.95 mm
  - SouthWest (SW): Max SW = 0.82 mm
  - SouthEast (SE): Max SE = 0.87 mm

- Anchor Load:
  - NorthWest (NW): Max NW = 4.33 kip
  - NorthEast (NE): Max NE = 1.78 kip
  - SouthWest (SW): Max SW = 2.74 kip
  - SouthEast (SE): Max SE = 2.88 kip

- Anchor Disp:
  - NorthWest (NW): Max NW = 0.11 in
  - NorthEast (NE): Max NE = 0.14 in
  - SouthWest (SW): Max SW = 0.03 in
  - SouthEast (SE): Max SE = 0.05 in

The graphs show the crack width in inches and millimeters, anchor load in kips and kN, and anchor displacement in inches and millimeters, with time in seconds.

Legend:
- Blue: NorthWest
- Green: NorthEast
- Yellow: SouthWest
- Red: SouthEast
- Solid line: Monotonic Uncracked
- Dashed line: Monotonic Cracked

The graphs also display the crack width in inches and millimeters, anchor load in kips and kN, with a color scale indicating the crack width.
STRUCT_N9_CC08_CR14_D14b  
AnchorType: Undercut

Max Accel = 0.61 \, g 

Abs. Accel.  
Max Disp = 0.01 \, in 

Abs. Disp.  
Max WALLE Accel = 1.12 \, g 

Abs. Accel. @ WALLE CM 
Max WALLE Disp = 0.49 \, in 

Abs. Disp. @ WALLE CM 

Time, sec 

Spectral Accel, g 

Period, sec 

WALLE Drift Ratio, % 

WALLE Accel, g 

WALLE Disp, in 

WALLE Drift Ratio, %
STRUCT_N9_CC08_CR14_D14b
AnchorType: Undercut
STRUCT_N9_CC08_CR14_D14c
AnchorType: Undercut

Slab Accel

Abs. Accel.
Max Accel = 0.64 g

Slab Disp

Abs. Disp.
Max Disp = 0.01 in

WALLE Accel

Abs. Accel. @ WALLE CM
Max WALLE Accel = 3.12 g

WALLE Disp

Abs. Disp. @ WALLE CM
Max WALLE Disp = 1.54 in

Time, sec

Spectral Accel, g

2% Damping

Period, sec

WALLE Drift Ratio, %

WALLE Accel, g

WALLE Drift Ratio, %
REFERENCES


**ACI 318 (2008)**  ACI 318 Committee (2008). “Building code requirements for structural concrete (ACI 318-08) and commentary (ACI 318R-08)”. American Concrete Institute, Detroit, MI.

**ACI 355 (2007)**  ACI Committee 355, "Qualification of Post-Installed Mechanical Anchors in Concrete (ACI 355.2-07) and Commentary (ACI 355.2R-07)." American Concrete Institute, Farmington Hills, MI, 2007, 34 pp.


Bazant & Oh (1983)

Beeby (1990)

Bommer & Martinez-Periera (1999)

Broms & Lutz (1965)

Broms (1965)

Broms (1965)

Cannon (1981)

Clough & Penzien (2003)

CSA (2001)


Frosch (1999)  
Frosch, R.J., “Another Look at Cracking and Crack Control in Reinforced Concrete,” ACI Structural Journal, No. 96-S49, May-June, 1999, pp. 437-442.

Fuchs et. al (1995)  

Furche (1987)  
Furche, J., “Versuchseinrichtung zur Prüfung von in Rissen verankerten Dübeln und erste Versuche an Parallelrippkörpern (Test Setup for the Testing of Fasteners in Cracks and Initial Tests in Parallel Cracks),” Institut für Werkstoffe im Bauwesen, Universität Stuttgart, 1987, 27 pp. (in German)

Furche (1998)  

Gergely & Lutz (1968)  

Hoehler & Eligehausen (2008a)  

Hayhurst (1992)  

Hoehler & Eligehausen (2008b)  
<table>
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<th>Author(s)</th>
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</table>
ICBO-ER 1372 (2000)

International Conference of Building Officials, Evaluation Report, ICBO-ES 1372 (2000), ITW Ramset / Red Head Self-Drilling, Trubolt Wedge, and Multi Set II Concrete Anchors

Jirasek & Bazant (1994)


Kunnath et al. (1990)


Lotze (1987)


Mahrenholtz, C. (2009)


Mahrenholtz, P. (2009)


Martin & Schwartzkopf (1980)


Marzouk et al. (2010)

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<th>Author(s)</th>
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</thead>
</table>
Rieder (2005)  

Rieder et al. (2008)  

Rodriguez et al. (2001)  

SEAOSC (1997)  
Structural Engineers Association of Southern California, “Standard Method of Cyclic Load Test for Anchors in Concrete or Grouted Masonry,” SEAOSC, Whittier, California, 1997, 6 pp.

Silva (2001)  

Sippel & Rieder (2007)  

Schwarz & Richardson (1999)  

Tang & Deans (1983)  


