Design and Instrumentation of the 2010 E-Defense Four-Story Reinforced Concrete and Post-Tensioned Concrete Buildings

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The opinions, findings, and conclusions or recommendations expressed in this publication are those of the author(s) and do not necessarily reflect the views of the study sponsor(s) or the Pacific Earthquake Engineering Research Center.
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This study reports on a collaborative research on the design, instrumentation, and preliminary analytical studies of two, full-scale, four-story buildings tested simultaneously on the NIED E-Defense shake table in December 2010. The two buildings are similar, with the same height and floor plan; one building utilized a conventional reinforced concrete (RC) structural system with shear walls and moment frames, whereas the other utilized the same systems constructed with post-tensioned (PT) members. The buildings were subjected to increasing intensity shaking using the JMA-Kobe record until a near-collapse state was reached. This report summarizes design issues and design documents, and provides detailed information on the type and location of sensors used. Initial analytical studies conducted both in the Japan and U.S. to support the design strategy and instrumentation of the buildings also are documented. The intent of this report is to provide a resource document for post-test research and high-impact education and outreach efforts.
ACKNOWLEDGMENTS

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This report was motivated by the desire to document the importance of these tests and to disseminate the rationale behind this testing program to the broader earthquake engineering communities in Japan and the U.S., as well as other countries, and to highlight important objectives. The joint report also documents the extraordinary level of collaboration between Japanese and U.S. researchers studying the response and performance of reinforced concrete structures. This collaboration has been so incredibly fruitful that universally the authors desire to continue such joint efforts in the future for many years to come.

The authors’ wish to acknowledge all the participants within the Reinforced Concrete Group of the various NEES–E-Defense workshops held in recent years in Japan and the U.S. These meetings and the relationships that have developed between the meeting participants have been key in laying the foundation for continued strong research collaboration in the present and the future.

Any opinions, findings, and conclusions or recommendations expressed in this material are those of the authors and do not necessarily reflect those of the Japanese Ministry of Education, Culture, Sports, Science, and Technology, the U.S. National Science Foundation, or other individuals mentioned or who have participated in the workshops and meetings.
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1 Introduction

1.1 BACKGROUND

In the 1994 Northridge and 1995 Hyogo-ken Nanbu (Kobe) earthquakes, many older reinforced concrete (RC) buildings suffered severe damage, and some collapsed due to brittle failure of key structural elements. In general, buildings designed to newer standards—such as the 1981 amendments to Japanese Building Standard Law Enforcement Orders and the 1976 and later versions of the U.S. Uniform Building Code—performed well. Some newer U.S. buildings performed poorly due to substandard behavior of diaphragms, particularly in precast prestressed concrete parking structures and gravity systems. In both Japan and the U.S., although building response to strong ground shaking generally satisfied code requirements and performed adequately in providing life safety, high repair costs as a result of nonlinear behavior produced large member cracks and residual deformations.

As a result, new design approaches were developed that focused on defining deformation limits that can be used to assess both collapse safety and the impact of damage on repair costs and loss of building use (down time). In the U.S., these new approaches are documented in FEMA-356 report and by reports published by the Pacific Earthquake Engineering Research (PEER) Center and others. Damage observed from significant earthquakes often results in an evolution of design practice, as witnessed in the 1994 Northridge earthquake for structural steel buildings and in the 2010 Chile earthquake for reinforced concrete wall buildings. As well, there is continuous pressure to develop structural systems that allow for longer spans and more flexible floor plans using new materials or new systems, such as prestressed and post-tensioned (PT) concrete systems. These new systems often have attributes that are different from commonly used systems, where laboratory testing and experience in earthquakes of both components and systems have been used to assess
expected performance and to verify design approaches. For example, PT systems typically have low hysteretic energy dissipation capacity relative to reinforced concrete (RC) systems; however, this same attribute tends to limit residual deformations. Therefore, it is important to continuously assess the expected performance of buildings constructed using new codes and new systems via testing of large-scale components and full-scale buildings models subjected to realistic loading histories expected in both frequent and rare earthquakes.

1.2 OBJECTIVES AND SCOPE

A series of shaking table tests were conducted on essentially full-scale RC and PT buildings designed using the latest code requirements and design recommendations available in both Japan and the U.S. To assess performance in both moderate-intensity frequent earthquakes (service-level) and large-intensity very rare earthquakes (collapse-level), the buildings were subjected to increasing intensity shaking using the JMA-Kobe and Takatori records until a near-collapse state was reached. The tests were designed to produce a wealth of data on stiffness, strength, and damping over a large range of deformations to assess current codes and recommendations, and will be used to develop new analysis tools and design recommendations, and determine if limit states and fragility relations used in current performance-based approaches to limit repair costs and assess collapse are consistent with measured responses and observed performance. The tests also will provide a wealth of data to assess and improve existing analytical tools used to model RC and PT components and systems, as well as help to identify future research needs.

1.3 ORGANIZATION

This report is divided into four chapters. The first chapter includes a brief introduction and background, followed by a short summary of the overall research objectives from both U.S. and Japan perspective. Chapter 2 provides an overview of the two test buildings, including a summary of design requirements, construction materials, structural drawings, and specimen construction. Chapter 3 includes a detailed description of the instrumentation used for each test building. Chapter 4 provides a brief summary and conclusions, as well as an overview of planned future studies.
1.4 BRIEF LITERATURE REVIEW AND OVERALL RESEARCH OBJECTIVES

The lengthy planning process and extensive collaboration between U.S. and Japan researchers leading up to the December 2010 tests produced test buildings that were designed to provide vital and important behavior and design information for both the U.S. and Japan. Because design objectives/requirements and performance expectations are somewhat different between the U.S. and Japan, a more detailed description of specific research objectives is provided in the following sections. In Chapter 2, the final building designs are reviewed using ASCE 7-05, ACI 318-08, and ACI ITG 5.1-07 to provide detailed information on U.S. code provisions and design recommendations that were met or not met.

1.4.2 Overall Objectives

When the Japanese Building Standard Law Enforcement Orders was substantially updated in 1981, the guiding principles of the new code were to prevent damage in minor and moderate earthquakes and to prevent collapse in severe earthquakes. These principles are essentially the same as those embodied in U.S. codes at the time (e.g., the Uniform Building Code). However, observations based on the 1994 Northridge earthquake and the 1995 Kobe earthquake, as well as other moderate to strong earthquakes that have occurred in recent years near major urban cities in Japan, have revealed that many buildings became nonfunctional and nonoperational due to damage to non-structural systems even if the structural damage was light to moderate. Based on these experiences, new design approaches have emerged in the 1990s and 2000s that address both structural and non-structural damage over a wider range of hazard levels. These approaches, which differ from prescriptive codes such as Uniform Building Code or the International Building Code, are commonly referred to as performance-based approaches, since the objective is to provide a more rigorous assessment of building performance.

Performance-based design approaches also provide a means to communicate expectations of building performance to the general public, building owners, and government agencies. This dialogue is essential, as there is a perception among the general public that buildings, both in Japan and the U.S., are “earthquake proof.” This perception is inconsistent with the stated code objectives of collapse avoidance. The economic losses and societal
impacts associated with buildings designed with current prescriptive code requirements are likely to be very significant, potentially impacting the affected region for many years.

Novel approaches have emerged to provide improved performance, for example, approaches that utilize response modification such as base isolation or using dampers. Although these approaches may offer excellent performance, in general, initial costs are high and other challenges exist (for base isolation one significant hurdle is accommodating the relative movement between the superstructure and the surrounding foundation, including utilities). Consequently, only a limited number of buildings are constructed utilizing these approaches.

Therefore, it is essential to continue developing performance-based approaches in conjunction with innovative cost-effective building systems that are capable of better performance relative to conventional construction. The RC and PT Buildings that are described in Chapter 2 were designed and the test protocol developed to provide vital information to address both of these issues. In the following three subsections, more detailed descriptions of test objectives are provided.

1.4.2 Test Building Specific Objectives

1.4.2.1 Performance-Based Seismic Design and Evaluation

Application of performance-based seismic design (PBSD), or performance-based seismic evaluation (PBSE), e.g., based on the PEER framework, has become fairly common. At a minimum, two hazard levels are considered: one associated with fairly frequent earthquakes with a return period of 25 or 43 years (a service-level event), and one associated with very rare earthquakes with a return period of approximately 2500 years (the Maximum Considered Earthquake, or MCE). A comprehensive PBSE might consider many hazard levels, e.g., ATC-58 [ATC 2007] considers 11.

Although relatively complex nonlinear modeling approaches are used to model frame and wall buildings, there is a lack of field and laboratory data available to assess the reliability of these models. With respect to shake table testing, data are mostly available for simple systems with one or two bays and one or two stories, often for effectively two-dimensional, moderate-scale structures utilizing a single lateral-force-resisting system (references) and
without gravity-load-resisting systems/members. The test buildings described Chapter 2 and 3 are essentially full-scale, three-dimensional buildings with different lateral-force resisting systems in the orthogonal directions. The availability of detailed measured response data along with observed damage will enable comprehensive system-level studies to assess the following issues: (i) the ability of both simple and complex nonlinear models to capture important global and local responses, including system interactions, both prior to and after loss of significant lateral strength; (ii) the capability of existing modeling approaches to capture loss of axial-load-carry-capacity (collapse); and (iii) the reliability of proposed PBSE approaches for new buildings (e.g., ATC-58) to predict the degree and distribution of damage and the related repair costs, as well as the margin against collapse for very rare events (e.g., MCE or higher level shaking).

1.4.2.1 High-Performance Building with Bonded RC Frame and Unbonded Post-tensioned Walls

One approach that improves a building’s performance is self-centering structural systems that utilize unbonded prestressed tendons. Initial research, conducted as part of the U.S. National Science Foundation’s (NSF) PREcast Seismic Structural Systems (PRESSSS) program in the 1990s [Shiohara 2001; Zhao and Sritharan 2007; Priestley 1991] demonstrated that such systems sustained relatively low damage compared to conventional RC systems under similar loading. This system has been implemented in a 39-story building in California [Priestley 1996] and for bridges [Priestley et al. 1999]. The self-centering framing system tested by the PRESSSS program involved relatively complex beam-column connection details. Subsequent research has been conducted to develop alternative systems/details [Englekirk 2002] and to extend the concept to steel structures [Pampanin et al. 2006] and timber structures [Pampanin 2005].

Primary research on self-centering systems in Japan began in 2000, with tests on hybrid column-beam joints with unbonded prestressing tendons and mild steel inside members by Sugata and Nakatsuka [2004], which was similar to the U.S. hybrid column-beam joint system. Sugata and Nakatsuka also proposed a numerical model [2005] to simulate flag shape hysteresis behavior exhibited by these connections, and Niwa et al. [2005] studied unbonded PT precast column-beam joint with external damping devices under the beam. Ichioka et al.
tested PT precast concrete portal frames with a corrugated steel shear panel placed between the beam and the foundation beam.

As shown in Figure 1.1, shake table testing has been conducted on reduced-scale (25%), three-story PT frames with bonded and unbonded beams [Maruta and Hamada 2010]. Test results demonstrated that PT precast concrete frames were very ductile, yet only minor damage was observed for velocities less than 50 kine. However, due to the self-centering capability, the system displayed low energy dissipation capacity (no damping devices were used). Self-centering systems have been developed and tested for structural steel systems [Ikenaga et al. 2007; Ichioka et al. 2009]; these systems have not yet been used in practice because design procedures have not been established to satisfy the Japanese Building Standard. In addition, the initial cost for the self-centering system is higher than conventional RC systems, and the potential long-term benefits of the system have not been sufficiently studied to assess if the higher initial cost is justified.

![Figure 1.1 Elevation of the longitudinal frame [Ikenaga et al. 2007].](image)

In this study the PT concrete structure is denoted at the “PT Building.” The design of the building is based on typical Japanese practice, with grouted PT precast prestressed concrete structure for beams and columns and unbounded prestressed concrete shear walls to
provide energy dissipation. To adequately compare the response of the RC Building and the PT Building, it was mandatory that the PT Building be designed such that the lateral force capacity of the PT specimen be close to that of RC specimen (for scientific interest); note that the Japanese code requires that the PT Building have slightly larger lateral strength than the RC Building. The PT Building also used high-quality, high-strength concrete. The innovative energy dissipative device utilized in the PT Building—the unbonded PT shear wall—has been investigated previously (see discussion above), but they have not been used in practice in either Japan or the U.S.

1.4.2.3 Reinforced Concrete Building - Moment Frame Direction

The conventional RC building system (RC Building) was designed to satisfy typical seismic design practice in Japan, with the quantity and arrangement of longitudinal and transverse reinforcement conforming to the Building Standard Law Enforcement Order and AIJ Standard. Typical materials were used to construct the test specimen. Preliminary analytical results presented by U.S. researchers at the October 2009 meeting in San Francisco and at the March 2010 meeting in Tokyo indicated that the design also reasonably represented U.S. Special Moment Frame (SMF) construction in California. A detailed assessment of the RC Building relative to U.S. code provisions is presented in Chapter 2.

Reinforced concrete special moment-resisting frames (SMRF) are commonly used in seismic regions, particularly for low- to mid-rise construction. Their behavior during seismic excitation depends on the behavior of individual members (e.g., columns, beams, joints, and slabs) and the interaction between members. Although numerous component tests have been performed on RC columns [Berry et al. 2004], beam-column joints and slab system tests that capture the interaction between these elements are rare [e.g., Ghannoum 2007; Panagiotou 2008]. Even less common are system tests that account for multi-directional dynamic loading effects. The E-Defense tests will help fill the knowledge gap in this area.

The influence of beam-column joint behavior on performance of the RC Building was identified as a topic of interest that could be assessed with the test buildings. Because test data within this range were not well represented in the literature and this range of strength ratios is common in Japan, Hiraishi et al. [1988] conducted quasi-static tests on beam-column joints with column-to-beam strength ratios between 1.0 and 2.0. The test results indicated that the
beam-column joint specimens performed uniformly poor, with significant strength loss and severely pinched hysteresis behavior due to bar slip, even if the demand on the joint (from beam yielding) was less than the joint shear strength. Given this information, the RC Building was designed to have beam-column joints that satisfy the weak-beam strong-column concept, but with calculated column-to-beam strength ratios near 1.2 for interior joints and 1.6 for exterior joints, respectively. The objective was to assess the behavior of joints in a conventional design at full scale on the E-Defense shake table.

As the structural engineering field moves towards PBSD, it is increasingly important to accurately model the full nonlinear behavior of SMRFs. Many challenges arise in nonlinear dynamic simulation due to the complex interactions between members and the variability in member boundary conditions. Current key challenges in simulating the seismic behavior of SMRFs are summarized below:

(1) **Evaluating the “elastic” stiffness of all members**: Structural stiffness is crucial for obtaining the correct seismic demand. Member stiffness is variable during seismic excitation and largely depends on axial load and level of cracking [Elwood and Eberhard 2009]. Element interactions also play a vital role. For example, strain penetration of longitudinal bars of columns and beams into joints and foundations can affect the stiffness of a structure by as much as 40% [Sezen and Setzler 2008; Zhao and Sritharan 2007]. Strain penetration effects in joints are highly dependent on joint demands and confinement, which can only be obtained from system tests.

(2) **Evaluating the strength of each member at which its behavior softens significantly**: In SMRF that strength usually coincides with the yield strength. It is particularly critical to achieve a model with the correct ratios of member strengths so that correct mechanisms are determined. While member yield strength can be estimated with reasonable accuracy for individual columns and beams, it is quite difficult to assess that strength in complete structural systems, particularly for monolithic beam/slab systems and joint construction. Quantifying the contribution of the slab on beam and joint capacities as well as the effect of strain rate effect under dynamic excitation is an especially important challenge that requires full system tests.
(3) *Simulating the post-“yield” response of each member:* Dynamic tests that cycle a structural system to very large deformations are necessary to obtain information about post-yield behavior. Structural assessment for the collapse prevention performance objective requires the identification of the deformation at which strength degradation is initiated and the ensuing degrading behavior. Such degradation can be the result of bar buckling, loss of shear strength, and fracture of transverse reinforcement in SMRF. Loading history and load sharing between structural elements both affect the initiation and the propagation of damage in elements. If adjacent elements are able to redistribute loads the behavior of the failing elements is significantly altered [Ghannoum 2007; Elwood and Moehle 2008]. Component tests cannot capture such system effects.

(4) *Simulating joint deformations and their progression during seismic excitation:* As with strain penetration effects, joint deformations can significantly affect the lateral stiffness of a SMRF. The joint-softening effect is particularly high at large deformations where joint damage can be substantial. The difficulty in assessing joint behavior stems from the fact that slabs, beams, and columns affect their behavior substantially. The beam-to-column strength ratio has particular influence on joint behavior [Shiohara 2001] as does bi-axial loading.

(5) *Assessing bi-axial loading effects on columns:* very few column tests are performed under bi-axial loading and even fewer dynamically. Bi-axial loading affects column strength as well as strength degradation.

1.4.2.4 *Reinforced Concrete and Post-tensioned Buildings - Shear Wall Directions*

Common Japanese practice uses columns at wall boundaries that are wider than the wall web (so-called barbell-shape). Over the past twenty years in the U.S., however, it has become common practice to design walls with rectangular cross sections. (Based on test results available in the literature, the AIJ Standard for “Structural Calculations of Reinforced Concrete Buildings” was revised in 2010 to show RC walls with rectangular cross section.) Although the deformation capacity attributed to wall shear failure or wall bending compression failure can be estimated using the "AIJ Design Guide Lines for Earthquake
Resistant Reinforced Concrete Buildings Based on Inelastic Displacement Concept," these procedures can be applied to walls with rectangular cross sections. Therefore, walls with rectangular cross sections were used in both the RC and PT Buildings to assess wall behavior at full-scale under dynamic loading. Primary objectives of the tests were to assess the behavior and performance of shear walls with rectangular cross sections to provide data to assess common practice in the U.S. and to potentially change practice in Japan, as well as to enable a side-by-side comparison between the conventional RC walls and high-performance PT walls.

Behavior and modeling of shear walls has received increased attention in recent years because not only do shear wall systems provide substantial lateral strength and stiffness, they are resilient to complete collapse [Wallace et al. 2008; EERI Newsletter 2010]. Recent testing conducted within the NEES-Research program includes quasi-static testing at: (i) nees@UIUC on isolated cantilever walls with rectangular cross sections with and without lap splices by Lowes and Lehman; (ii) nees@Minnesota on isolated, cantilever walls with both rectangular and T-shaped cross sections subjected to uniaxial and biaxial loading by French and Sritharan, and (iii) nees@UCLA by Wallace and nees@Buffalo by Whittaker on low-to-moderate aspect ratio (one to two), isolated walls with rectangular cross sections. Shake table tests on very-large scale, eight-story walls with both rectangular and T-shaped cross sections subjected to uniaxial loading have been conducted at nees@UCSD (Panagiotos and Restrepo). Tests also have been conducted on PT walls (Sause and others). Therefore, the full-scale shake table tests on the RC and PT Buildings will provide a wealth of data, including information on shear wall systems (walls and frames) subjected to three-dimensional, dynamic loading.

Nonlinear modeling of shear walls has been the subject of much research in the last five years, with considerable attention has focused on modeling flexure-shear interaction, i.e., where yielding in shear is observed for relatively slender, isolated walls, with aspect ratios \( A_w = h_w/l_w \) between 2.4 (PCA tests) and 3.0 (e.g., see Massone and Wallace [2004]), even though the computed nominal shear strength exceeds the shear demand. The RC Building tested at E-Defense will provide important results for system level tests of slender walls \( A_w = 4.8 \) coupled by a shallow beam to corner columns at low axial load. The tests will provide data for a case where flexure-shear interaction is expected to be minor. Quasi-static tests are currently being conducted to assess flexure-shear interaction for moderate aspect
ratio walls \( A_w = 1.5 \) to \( 2.0 \) and quasi-static loading [Tran and Wallace 2010]; future shake table testing is needed to further address this need.

Slightly different detailing has been provided within the yielding regions (plastic hinge regions) of the shear walls on the north and south sides of the conventional RC building to investigate the role of detailing on damageability, lateral strength degradation, and, potentially, the loss of axial load carrying capacity. Given the likely role of detailing on the observed damage in the recent \( M_w 8.8 \) February 27, 2010, earthquake in Chile, this aspect of the test is of significant interest.

The impact of modest coupling on lateral story displacements and wall shear forces has not yet been studied, particularly for dynamic loading of three-dimensional building systems. The E-Defense tests will provide a wealth of data to assess these issues, as well as the increase in wall shear with shaking intensity.
2 Test Buildings

Descriptions of the RC and PT buildings are provided in the following sections. Background information is provided on the E-Defense shake table and detailed information on overall geometry, member dimensions, and longitudinal and transverse reinforcement are presented for the RC and PT buildings.

2.1 BACKGROUND

The E-Defense shake table, the largest in the world, has plan dimensions of 20 m × 15 m (Figure 2.1). The table can produce a velocity of 2.0 m/sec and a displacement of 1.0 m in two horizontal directions, simultaneously, and accommodate specimens weighing up to 1200 metric tons. In this study, two four-story buildings were tested, one RC and one PT. The two buildings were almost identical in geometry and configuration, and were tested simultaneously, as shown in Figure 2.2. Each building weighed approximately 5900 kN; therefore the combined weight of the two buildings was 98% of E-Defense table capacity. The test buildings utilized different structural systems to resist lateral forces in the longitudinal and transverse directions. In the longitudinal direction, a two-bay moment frame system was used, whereas in the transverse direction, structural (shear) walls coupled to corner columns by slab-beams were used at each edge of the buildings (Figure 2.3). Story heights at all levels for both buildings were 3 m, for an overall height of 12 m. The plan dimensions of the buildings were 14.4 m in the x- or frame direction and 7.2 m in the y- or wall direction.
Figure 2.1 E-Defense shaking table.

Figure 2.2 Overview of test set up on the shaking table.
2.2 REINFORCED CONCRETE BUILDING

Plan and elevation views of the structure are shown in Figure 2.3 and Figure 2.4, respectively. Cross-section dimensions of columns were 500 mm × 500 mm, and walls were 250 mm × 2500 mm; beam cross-sections were 300 mm × 600 mm (width × depth) in the x-direction and 300 mm × 400 mm for interior beams and 300 mm × 300 mm for exterior beams in the y-direction. Additional beams with cross sections of 300 × 400 mm supported the floor slab at intervals of 1.5 m in the y-direction. A 130 mm-thick floor slab was used at floor levels 2 through 4 and at the roof level. Detailed information on member geometry and reinforcement used is given in Appendix A.2. Information on the building weight and material properties are contained in Table 2. and Table 2., respectively. Building weight was calculated based on the design, i.e. before the non-structural members were placed in the specimens. Floors 2 through 4 weighed about 900 kN, whereas the weight of the roof was 1000 kN; the remaining weight was in the foundation. The weight of the equipment is presented in Appendix A.1.

The design concrete compressive strength was 27 N/mm², with SD345 D19 and D22 bars used for primary longitudinal reinforcement. Information on the longitudinal and transverse reinforcement used in all members is provided in Table 2. and Figure 2.5. Typical concrete stress versus strain relations are given in Figure 2.6. See Appendix A.1 for detailed information on as-tested material properties.
<table>
<thead>
<tr>
<th></th>
<th>RC</th>
<th>2.4</th>
<th>t/m³</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>RFL</td>
<td>4FL</td>
<td>3FL</td>
</tr>
<tr>
<td>RC</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Column</td>
<td>5.4</td>
<td>10.8</td>
<td>10.8</td>
</tr>
<tr>
<td>Girder</td>
<td>16.4</td>
<td>16.4</td>
<td>16.4</td>
</tr>
<tr>
<td>Wall</td>
<td>4.1</td>
<td>8.1</td>
<td>8.1</td>
</tr>
<tr>
<td>Slab</td>
<td>44.1</td>
<td>43.7</td>
<td>43.3</td>
</tr>
<tr>
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<td>8.0</td>
<td>8.0</td>
</tr>
<tr>
<td>Parapet</td>
<td>5.3</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>Steel</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Temp. Girder</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>Sum [t]</td>
<td>83.3</td>
<td>87.0</td>
<td>86.6</td>
</tr>
</tbody>
</table>

| Non-Structural            |         |       |       |       |       |
| Steel                     | Stair   | 330   | 360   | 360   | 360  | 0   |
|                          | Measurement | 0     | 3000  | 1750  | 1690 | 1690|
|                          | Handrail | 244   | 271   | 271   | 271  | 197 |
| Machine on the slab       | 4633    | 180   | 0     | 0     | 0   |
| under the slab            | 495     | 0     | 0     | 0     | 0   |
| RC Base                   | 6042    | 346   | 0     | 0     | 0   |
| Ceiling under the slab    | 296     | 0     | 0     | 0     | 0   |
| Sum [kg]                  | 12040   | 4157  | 2381  | 2321  | 1887|

| Total                     |         |       |       |       |       |
| Sum                       | 95.3    | 91.2  | 89.0  | 88.5  | 238.4|
| Whole Building [t]        | 602.4   |
Table 2.2  Design material properties.

<table>
<thead>
<tr>
<th></th>
<th>(a) Concrete</th>
<th>(b) Steel Bar</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\sigma_B$ (N/mm²)</td>
<td>Grade</td>
</tr>
<tr>
<td>Foundation</td>
<td>33</td>
<td>D22 SD345</td>
</tr>
<tr>
<td>Upper Part</td>
<td>27</td>
<td>D19 SD345</td>
</tr>
<tr>
<td></td>
<td></td>
<td>D13 SD295</td>
</tr>
<tr>
<td></td>
<td></td>
<td>D10 SD295</td>
</tr>
<tr>
<td></td>
<td></td>
<td>D10 KSS785</td>
</tr>
</tbody>
</table>

Figure 2.4  Elevation view of specimens.

Figure 2.5  Reinforcement stress-strain relations.
Figure 2.6  Concrete stress-strain relations.

2.2.1 Japanese Standard Law Provisions

The RC buildings were designed to conform to the Japanese Building Standard Law. The Japanese seismic design procedure consists of two stages design; allowable stress design for moderate earthquake level to guarantee the damage control performance, and lateral load capacity design for major to rare earthquake to guarantee the collapse prevention performance.

The base shear coefficient $C_b$ for the allowable stress design is 0.20. The lateral force distribution shape is an $A_i$ distribution, which is similar to inverted triangular where the lateral load at the top-most stories is slightly larger. For the structural analysis, the building was modeled as linearly elastic. All member response was designed to not exceed the yielding level for reinforcing bars, and the concrete stress response was designed to not exceed the allowable compressive stress of concrete—two third of concrete design strength.

The design base shear coefficients $C_b$ for the lateral load capacity at collapse mechanism of the conventional RC Building were 0.30 in the frame direction and 0.35 in the wall-frame direction, respectively, as all structural members were designed to perform at the
highest possible ductility. The lateral capacity of the building was confirmed by pushover analysis that considered nonlinear material characteristics; the lateral force distribution shape $A_i$ was used. Capacity design checks were carried out for shear failure of beams, columns, and shear walls, as well as shear failure of beam-column joints; note that there was no requirement regarding the column-to-beam strength ratio at the beam-column joints. Shear reinforcement provided in columns and beams (in the moment frame or $x$-direction) and walls (in the $y$-direction) had shear reinforcement in excess of that required by the Japanese Building Standard Law. Minimum requirements such as the spacing of the steel, anchorage detail, dimension of concrete section as well as concrete cover thickness were designed in accordance with the AIJ Standard for reinforced concrete structures. Thus the RC Building accurately represented a building that followed typical construction practices common in Japan.

### 2.2.2 Assessment of RC Building using ASCE 7-05 and ACI 318-08

A detailed assessment of the RC Building was conducted to assess whether the final design satisfied U.S. code provisions. This assessment is covered in two subsections—one for the shear wall direction and one for the moment frame direction—to provide the reader with information to help understand the measured responses and observed behavior once this information becomes available.

#### 2.2.2.1 Shear Wall Direction

For the shear wall ($y$-) direction, the structural system was assumed to be a Building Frame System Special RC Shear Wall ($R = 6, C_d = 5$) as the framing provided by the shallow beam and column at the building edge was insufficient for a Dual System designation. Based on this designation, all lateral forces are resisted by the shear wall. Given that the building system is relatively simple, the ASCE 7-05 S12.8 Equivalent (Static) Lateral Force Procedure was used, assuming that the building was located in a region where the mapped short period and 1-sec-period accelerations were 1.5 and 0.9, respectively; for Site Class B, design spectral acceleration parameters were 1.0 and 0.6 with $T_o = 12$ and $T_s = 0.6$.

The seismic weight (ASCE 7-05, 12.7.2) of the building was taken as the combined dead and live loads as 3630 kN (see Table 2.1), i.e., the live load value includes permanent
live load attached to the building. The fundamental period of the building was computed using a two-dimensional model of a single wall, i.e., a cantilever assuming an effective moment of inertia $I_{eff} = 0.51g$ over the full wall height and one-half the seismic weight at the floor levels. A fundamental period of $T = 0.58$ sec was computed from an eigenvalue analysis. According to ASCE 7-05 12.8.2, $T_a = 0.488(h_a = 12 \text{ m})^{0.75} = 0.315$ sec $T_a$ and $T_s = C_sT_a = 1.4T_a = 0.0440$; therefore, $T = 0.44 = T_a$ was used to determine a base shear of $V = C_s = 0.167W = 302.5$ kN. Because only two shear walls were used—one at each end of the building—the redundancy factor (ASCE 7-05 12.3.4) was taken as 1.3. Therefore, $E_h = \rho Q_E = 1.3(302.5 \text{ kN}) = 393.3 \text{ kN}$ (ASCE 7-05 Equation 12.4-3). Vertical earthquake loading $E_v$ was included in the load combinations (ASCE 7-05 12.4.2 and 12.4.2.3).

**Strength Requirements for Walls:** Dead and live loads for the wall were calculated by assuming the dead and live loads (see Table 2.1) were uniformly distributed based on a tributary area equal to the wall length (2.5 m) plus the beam clear length (2.5 m + 2.1 m) times one-half the joist spacing and the slab overhand (0.9 m + 0.8 m), or 7.82 m$^2$ (84.2 ft$^2$). Shown in Figure 2.7, the resulting story forces produce wall base moment $M_u = 3569$ kN-m and axial load $P_u = 285$ kN. Note that the axial load ratio is low $\left[ P_u/A_yf_{y} = 285 \text{ kN}/(0.25 \text{ m} \times 2.5 \text{ m})(27 \text{ MPa}) = 0.017 \right]$. Demands were compared with a wall P-M interaction diagram (see Figure 2.8), demonstrating that the wall P-M strength does not satisfy ASCE 7-05 12.8 requirements.

**Capacity Design Checks:** Wall shear strength was computed as $\phi V_u = 0.75 A_y \left( \alpha_c \sqrt{f_{c}'} + \rho_t f_{y} \right) = 912 \text{ kN}$, using the minimum horizontal web reinforcing ratio (2D10 @ 200 mm spacing for the wall at axis $C$, $t_w = 250$ mm; $\rho_t = 0.0031$; $\alpha_c = 0.167$; $f_{c}^t = 27 \text{ MPa}$; $f_{y} = 345 \text{ MPa}$). Calculated shear strength $\phi V_u = 912 \text{ kN}$ is much greater than shear demand $V_u = 393$ kN, as would be expected given the relatively high wall aspect ratio (12 m/2.5 m = 4.8). The wall shear strength at axis $A$ is much larger as a result of the 125 mm spacing of the horizontal web reinforcement.
Figure 2.7  Equivalent lateral loads on the shear wall system.

Figure 2.8  P-M interaction diagram for the wall.
**Drift Requirements in the Wall:** Lateral displacements and story drifts were computed according to ASCE 7-05 12.8.6 and compared to allowable story drift per Table 12.12-1 where $0.02h_{ax}/ρ = 1.3 = 0.0154h_{ax}$. Story drift ratios of 0.0045, 0.0113, 0.0151, and 0.0167 were computed (Figure 2.9). The drift ratio for the fourth level exceeded the ASCE 7-05 limit by 8% ($0.0167/0.0154 = 1.08$).

**Detailing Requirements in the Wall:** Detailing requirements at wall boundaries were checked using the displacement-based approach of ACI 318-08 21.9.6 (21.9.6.2); the roof drift ratio $\left(\frac{\delta_u}{h_u} = 0.142/12 \text{ m} = 0.012\right)$ exceeded the minimum value of 0.007. Based on this value, the critical neutral axis depth using ACI 318-08 equation (21-8) is 352 mm. The neutral axis depth computed for the given wall cross section for an extreme fiber compression strain of 0.003 with $P_u = 285 \text{ kN}$ is 244 mm; therefore, special boundary elements are not required per 21.9.6.2. The vertical reinforcing ratio of the boundary reinforcement $[\rho = 6A_b/(2x+a) = 0.017, \text{ with } A_b = 284 \text{ mm}^2, \text{ h} = 250 \text{ mm, } (2x+a) = 400 \text{ mm}],$ exceeded $\rho = 2.3/f_y = 0.0067$, where $f_y = 345 \text{ MPa}$; therefore, ACI 318-08 21.9.6.5(a) must be satisfied as a hoop spacing cannot exceed 203 mm. The configuration and the spacing used at the wall boundary satisfies the requirements of 21.9.6.5(a), since the spacing of hoops and crossties is 80 mm (axis $A$) and 100mm (axis $C$), and a hoop and a crosstie are provided (all 6 bars are supported) over a depth of almost 400 mm, which significantly exceeds the minimum depth required from 21.9.6.4(a) of one-half the neutral axis depth (244 mm/2).

If the “stress-based” approach of 21.9.6.3 is used, however, the extreme fiber compression stress of $f_c = M_u/s + P_u/A = 11.56 \text{ MPa} \quad (M_u = 3569 \text{ kN-m}; P_u = 285 \text{ kN}; I_g/S = 0.26 \text{ m}^3; \text{ and } A_g = 0.625 \text{ m}^2)$ significantly exceeds the stress limit of $0.2f'_c = 5.4 \text{ MPa}$, with 21.9.6.4 left to be satisfied and requiring special boundary elements. Based on a wall boundary zone with $b_{cx} = 160 \text{ mm}, \ b_{cy} = 320 \text{ mm, } A_{shx} = 2A_b, A_{shy} = 3A_b, \ A_h = 78.5 \text{ mm}^2, \ s = 80 \text{ mm (axis A) or 100 mm (axis C)}, \ f'_c = 27 \text{ MPa}, \text{ and } f_{yr} = 345 \text{ MPa},$ the provided $A_{sh}$ values are 1.39 and 2.09 times that required by ACI 318-08 Equation (21-5) for 100 mm spacing, satisfying 21.9.6.4. Note that the provided $A_{sh}$ values are only 0.45 and 0.34 times that required by ACI 318-08 Equation (21-4).
In summary, the RC shear wall generally satisfies ASCE 7-05 and ACI 318-08 requirements for the assumed design spectrum, although the wall P-M strength does not meet the requirement and the interstory drift ratio in the top floor exceeds the limiting value by 8%.

(see Figure 2.9).

2.2.2.2 Frame Direction

For the frame (x-) direction, the structural system was assumed to be a Special Reinforced Concrete Moment Frame \( (R = 8, C_d = 5.5) \), whereby the lateral forces are resisted by a four-story, two-bay frame at the perimeter of the building.

![Interstory drift demands for the wall.](image)

**Figure 2.9** Interstory drift demands for the wall.

The fundamental period of the building was computed using a two-dimensional model of a single perimeter moment frame, assuming an effective moment of inertia \( I_{\text{eff}} = 0.3I_g \) for beams and columns (based on ASCE-41) and one-half the seismic weight at the floor levels. A fundamental period of \( T = 0.67 \text{ sec} \) was computed from an eigenvalue analysis. According to
ASCE 7-05 12.8.2,  \( T_a = 0.0466(h_n = 12 \text{ m})^{0.9} = 0.44 \text{ sec} \) and  \( T_u = C_u T_a = 1.4T_a = 0.610 \); therefore,  \( T = 0.56 = T_u \) was used to determine a base shear of  \( V = C_s W = 0.125W = 226.9 \text{ kN} \). The redundancy factor (ASCE 7-05 12.3.4) was taken as 1.3, since the structure was expected to have an extreme torsional irregularity by loss of moment resistance at the beam-to-column connections at both ends of a single beam (which is the worst case scenario); therefore,  \( E_u = \rho Q_e = 1.3(226.9 \text{ kN}) = 294.9 \text{ kN} \) (ASCE 7-05 Equation 12.4-3). Vertical earthquake loading  \( (E_v) \) was included in the load combinations (ASCE 7-05 12.4.2 and 12.4.2.3).

**Strength Requirements for Beams and Columns:** Dead and live loads for the beams and columns—calculated by assuming the dead and live loads (see Table 2.1)—were uniformly distributed based on a tributary area associated with the member, e.g., for the corner column this is equal to approximately one-eighth the entire floor plan minus one-half the wall tributary area, or 18.1 m² (81 ft²) (see Figure 2.10). Using the same spectral acceleration parameters and seismic weight that were used in the shear wall system calculations, the ASCE 7-05 S12.8 Equivalent (Static) Lateral Force Procedure was used; the resulting story forces are shown in Figure 2.11. These forces were applied to the two-dimensional model to compute the member demands. At the base of the first story, columns values were computed to be  \( M_u = 205 \text{ kN-m} \) and axial load  \( P_u = 772 \text{ kN} \) for the corner columns (C1), and  \( M_u = 200 \text{ kN-m} \) and  \( P_u = 1222 \text{ kN} \) for the interior column (C2). Note that the axial load ratio was  \( P_u/\phi f' = P_u = 772 \text{ kN}/(0.5 \text{ m} \times 0.5 \text{ m})(27 \text{ MPa}) = 0.11 \) for the corner columns and 0.18 for the interior column.
Figure 2.10 Tributary area for corner column C1.

Figure 2.11 Equivalent lateral loads on the frame system.

Beam and column nominal moment capacities were computed, and the column, beam, and joint shear demands computed to assess if the system satisfied capacity design concepts that promote beam yielding. Slab effective widths were based on the provisions of ACI 318-08 8.12. Calculation details are provided in Appendix B. The concrete stress-strain relation was assumed to have a peak of 27 MPa (3.9 ksi) at 0.002 strain, and the steel stress-strain relation was assumed as an elastic-perfectly plastic behavior with a yield strength of 345 MPa (50 ksi) and an ultimate strength of 490 MPa (71 ksi). Moment and axial load demands of the columns were compared with a column P-M interaction diagram (Figure 2.12) and for the
corner column (C1) (Figure 2.13) and the interior column (C2), respectively. The results demonstrate that the column P-M strengths satisfy ASCE 7-05 12.8 requirements.

In addition, beam moment demands were checked in accordance with the provisions of ACI 318-08 S21.5 such that \( M^+ > M^- / 2 \), and neither negative or positive moment strength at any section along the member length was less than one-fourth the maximum moment strength at the face of either joint. The amount of reinforcement in the beams was \( A_{s,provid} = 1140 \text{ mm}^2 \) \( (\rho_{provid} = 0.007) \), which is much greater than the minimum required reinforcement per ACI 318-08 S21.5.2, \( A_{s,min} = 654 \text{ mm}^2 \), and less than the maximum allowed reinforcement ratio \( \rho_{max} = 0.025 \). The reinforcement was continuous along the entire span, indicating that beam moment strengths satisfy the provisions of ACI 318-08 21.5.

### Figure 2.12 P-M interaction diagram for corner column C1.
Capacity Design Checks

Columns Shear Strength (21.6.5): Beam shear demands were determined as when beam probable moment strength was reached (calculated using $f_s = 1.25 f_y$), column shear when column probable moments were reached, and beam probable moments reached for the interior, first-story column [see Figure 2.14(a)] and a typical beam [Figure 2.14(b)]. Nominal shear strengths also are shown, demonstrating that beam and column shear strengths were sufficient to develop the beam probable moments, and the column shear strength was sufficient to resist the column shear developed at column probable moments.

Beam Shear Strength (21.5.4): ACI 318-08 requires that beams of special moment frames be designed such that flexural yielding occurs prior to shear failure. Therefore, beam shear strengths were checked to sufficient capacity to resist the shear that develops when the beam reaches its probable moment of flexural capacity at each end (see Figure 2.15). The demand calculation was based on the gravity loading on the beams and beam probable moments. Shear demand and capacity in the beams are also shown in Figure 2.15. Results of
this assessment are shown in Figure 2.13, demonstrating that beam shear strength satisfied ACI 318-08 requirements for a special moment frame.

**Strong-Column Weak Beam (21.6.2):** The strong column–weak beam provision of ACI 318-08 was checked at all floor levels; this requires that sum of column nominal moment strength $\sum M_{nc}$ be at least 1.2 times the sum of the beam nominal moment strengths $\sum M_{nb}$. Column flexural strengths were calculated with the factored axial force, resulting in the lowest strength [where $(0.9-0.2S_{DS}) D + \rho E)]$. Beam nominal strengths were calculated including an effective slab width per ACI 318-08 8.12. Results presented in Figure 2.16 demonstrate that corner columns satisfy these requirements, whereas interior columns have the column-to-beam strength ratios about 1.0 ($< 1.2$). Note that the ratio at the roof level connections is smaller than 1.0, indicating that column yielding might occur at the roof level.

The design of beam-column joints was calculated according to ACI 318-08, Section 21.7, defined as: (1) joint shear demand $V_u$, (2) joint nominal shear strength $\phi V_n$, (3) required transverse reinforcement; and (4) required anchorage. Next, each of these parameters are assessed to determine whether or not the given requirements are satisfied for an interior connection (case 1: G1-C2-G1), and for an exterior connection (case 2: G1-C2). Additional details and information for other connections are provided in Appendix B.

\[
M_{pr, col} = 1.25(M_{n, col} = 486) = 607kNm
\]

\[
M_{pr, b} = 1.25(M_{n, b} = 386) = 483kNm
\]

\[
M_{pr, h} = 715kNm
\]

\[
V^{(1)}_E = \frac{2M_{pr, col}}{h} = \frac{2(607kNm)}{h = 2.4m} = 506kN
\]

\[
V^{(2)}_E = \frac{M_{pr, b} \pm M_{pr, h}}{h} = \frac{1198kNm}{h = 2.4m} = 499kN
\]

\[
\phi V_N = (0.75)V_N = 675 = 506kN
\]

*Figure 2.14  Column shear strength demands.*
\[ w = 10.4\text{kN/m} \]

\[ M_{pr,b}^+ = 482.1\text{kNm} \]

\[ M_{pr,b}^- = 715.3\text{kNm} \]

\[ [V_{u,pr}]_R = [V_{u,pr}]_{\text{max}} = \frac{M_{pr,b}^+ + M_{pr,b}^-}{l} + \frac{w_g l}{2} = 214\text{kN} \]

\[ \phi V_N = (0.75)(V_N = 289) = 217\text{kN} \]

**Figure 2.15**  Beam shear strength demands.

**Figure 2.16**  Column-to-beam strength ratios.
Given the weak-beam requirements and capacity design requirements for beam and column shear, beams that frame into beam-column joints are typically assumed to yield prior to the columns. Therefore, the demands on the joint are controlled by the quantity of longitudinal reinforcement used in the beams, as well as the stress developed in these bars. In ACI 318-08 S21.5.4, the probable moment is calculated for a minimum longitudinal reinforcement stress of 1.25$f_y$. Joint shear demand for both cases was calculated using horizontal joint equilibrium (Figure 2.17) resulting in:

$$V_{u,\text{joint},1} = 1.25A_{s,b}f_y + 1.25A_{s,b}2f_y - V_{c1}$$

for an interior connection (case 1), and

$$V_{u,\text{joint},2} = 1.25A_{s,b}2f_y - V_{c1}$$

for an exterior connection (case 2). Here, $V_{c1}$ represents the column shear, which can be estimated as:

$$V_{c1} = M_{c1}/(h_{\text{clear}}/2)$$

where $M_{c1} = M_{c2} \approx (M_{pr,b1} + M_{pr,b2})/2$ for case 1, and $M_{c1} = M_{pr,b1}/2$ for case 2. According to Section 21.7.4, joint shear demands for case 1 and case 2 are $V_{c1,1} = 936$ kN and $V_{c1,2} = 538$ kN, respectively. Using values of $\phi_v = 0.85$, and $\gamma_v = 12$ (for both cases), the joint shear capacities calculated according to Section 21.7.4 are: $\phi V_{u,1} = \phi V_{u,2} = 1097$ kN. Note that the nominal shear capacities are greater than shear demands.

![Figure 2.17](image-url)  
Figure 2.17  Free body diagrams for (a) interior and (b) exterior beam-column connection.
Drift Requirements in the Frame: Lateral displacements and story drifts were computed according to ASCE 7-05 12.8.6 and compared to allowable story drift per ASCE 7-05 Table 12.12-1 of \(0.02h_{sv}/\rho = 1.3 = 0.0154h_{sv}\). As was done to determine the fundamental period, effective moment of inertia values of \(0.3I_g\) were used for the beams and columns based on ASCE 41-06 recommendations. Story drift ratios of 0.0099, 0.0134, 0.0108, and 0.0068 were computed, and, the drift ratios did not exceed the ASCE 7-05 limit (Figure 2.18).

Detailing Requirements: Detailing requirements for columns were compared with ACI 318-08 S21.6.4 provisions. Spacing of the transverse reinforcement in the columns was compared with the ACI 318-08 S21.6.4.3 provisions where the minimum required transverse reinforcement spacing is:

\[
s_{\text{min}} = \min(h/4 = 125 \text{ mm}; \ 6d_{tb} = 132 \text{ mm}; \ s_o = 140 \text{ mm}; \ 6 \text{ in.} = 152.4 \text{ mm}) = 125 \text{ mm}
\]

where \(s_o = 4 + (14 - h_x)/3\) and \(h_x = 240 \text{ mm}\) Using ACI 318-08 S21.6.4.4, the minimum required spacing was also calculated to provide the transverse reinforcement. For example, for
the interior column at the base, transverse reinforcement quantity was obtained as

\[ A_{sh} = 4A_b = 314 \text{ mm}^2, \quad A_b = 78.5 \text{ mm}^2, \quad s_{\text{min}} = 73 \text{ mm} \quad (\text{ACI 318 21-4}) \]

and

\[ s_{\text{min}} = 107 \text{ mm} \quad (\text{ACI 318 21-5}), \quad f_c = 27 \text{ MPa}, \quad f_y = 345 \text{ MPa}, \quad b_c = 417 \text{ mm}, \]
\[ A_g = 250,000 \text{ mm}^2, \quad A_{ch} = 417^2 \text{ mm}^2. \]

\[
s_{\text{min}} = \left( \frac{A_{sh}}{0.3b_c f'c \left( \frac{A_g}{A_{ch}} - 1 \right)} \right) = 73 \text{ mm} \quad \text{Eq. (1) (ACI 318 21-4)}
\]

\[
s_{\text{min}} = \left( \frac{A_{sh}}{0.09b_c f'y} \right) = 107 \text{ mm} \quad \text{Eq. (2) (ACI 318 21-5)}
\]

Therefore, the spacing provided in the column \((s = 100 \text{ mm})\) satisfies all spacing requirements except \(s_{\text{min}} = 73 \text{ mm}\) determined from (Eq. 21-4). This spacing requirement is not satisfied either at the other floors or in the corner columns. Note that the required transverse reinforcement should be based on these limits within a height of \(l_o\), which is

\[ l_o = \min (h = 500 \text{ mm}; 1/6h_{\text{clear}} = 400 \text{ mm}; 18 \text{ in.} = 152.4 \text{ mm}) = 400 \text{ mm} \quad \text{(see Figure 2.19).}
\]

Beyond \(l_o\), ACI 318 limits the spacing to

\[ s_{\text{min}} = \min (6d_{ht} = 132 \text{ mm}; 6 \text{ in.} = 152.4 \text{ mm}) = 132 \text{ mm} \]

therefore, beyond \(l_o\) (i.e., within the middle portion of the column height), ACI 318 requirements are satisfied because \(s = 100 \text{ mm}\) is used.

Detailing requirements at the beams also were checked using ACI 318-08 S21.5.3. Hoops are required over a length equal to twice member depth \((2h \text{ region} = 1200 \text{ mm})\) (see Figure 2.19). Minimum required spacing in this region was calculated as

\[ s_{\text{min}} = \min (d/4 = 150 \text{ mm}; 8d_{ht} = 176; 24d_{ht} = 240; 12 \text{ in.} = 304.8 \text{ mm}) = 150 \text{ mm} \]

which does not satisfy the provision, since the provided spacing is \(s = 200 \text{ mm}\). Beyond the \(2h\) region, where hoops are not required by ACI 318, minimum spacing is defined as

\[ s_{\text{min}} = d/2 = 273 \text{ mm} \quad \text{and is satisfied.} \]
Required transverse reinforcement in the beam-column joints is calculated according to Section 21.7.3.1. Since \( b_w < \frac{3}{8} b_{col} \), the required transverse reinforcement is 100% of \( A_{sh} \) computed for columns. This provision is not satisfied for the same reason as found in the case of columns (see detailed discussion in the previous section regarding this issue). Development length of bars in tension was calculated according to Section 21.7.5 \( l_{dh} = \frac{f_y d_b}{(65 (f'c)^{0.5})} \). For both cases of joints this provision is satisfied since the actual development length is greater than the required value.

![Diagram of beam-column joints](image)

**Figure 2.19** Locations where special hoop requirements are needed.

### 2.2.2.3 Collapse Mechanism

A collapse mechanism analysis was conducted for both the shear wall and moment frame directions using the code prescribed distribution of lateral forces over the building height. Four different collapse mechanisms were assumed for each direction: column yielding at the first, the second, the third, and the fourth floors. Figure 2.20 shows base shear calculated for each collapse mechanism assumption. For the moment frame, the expected collapse mechanism is beam hinging accompanied by hinging at the base of first floor columns and at the top of the second floor columns (Figure 2.21). For the shear wall direction, the mechanism involves beam hinging accompanied by yielding at the base of first floor walls (Figure 2.22). The actual strength coefficients are approximately 0.45 and 0.50 for the moment frame and
wall-frame directions, respectively, or 3.6 and 3.0 times the values given in ASCE 7-05. Note that the overstrength factors given in ASCE 7-05 Table 12.2-2 are 3.0 and 2.5 for the moment frame and shear wall, respectively. Therefore, the computed overstrengths for the wall and moment frame are higher than expected (3.6 versus 3.0 for frame and 3.0 versus 2.5 for shear wall direction).

Figure 2.20  Collapse mechanism assessment-influence of column yielding level.

Figure 2.21  Controlling collapse mechanism in the frame direction.
2.3 POST-TENSIONED BUILDINGS

Table 2.3 details the weight and material properties of the specimen. The weight of each floor from the second to the fourth floor was about 900 kN and the weight of roof floor was 1000 kN. The weight above the foundation was about 3700 kN. The design strength of the precast concrete was 60 N/mm². The plan is shown in Figure 2.3 and the elevation in Figure 2.4. The columns were 450 mm x 450 mm square, the walls 250 mm x 2500 mm thick, and the beams 300 mm x 500 mm in the longitudinal direction. The beam of interior frame was 300 mm x 300 mm in the transverse direction, and the beam of exterior frame was 300 mm x 300 mm. The floor slab was 130 mm thick. Beams 300 x 300 mm square supported the floor slab at intervals of 1.0 m in the transverse direction.
Table 2. Design material properties of post-tensioned specimen.

<table>
<thead>
<tr>
<th>STEEL</th>
<th>Grade</th>
<th>( A_{\text{normal}} ) (mm(^2))</th>
<th>( \sigma_y ) (N/mm(^2))</th>
<th>( \sigma_t ) (N/mm(^2))</th>
</tr>
</thead>
<tbody>
<tr>
<td>D22 (ED for wall base)</td>
<td>SD345</td>
<td>387</td>
<td>385</td>
<td>563</td>
</tr>
<tr>
<td>PT bar ( \phi 21 ) (1-3Fl column)*</td>
<td>C</td>
<td>346.4</td>
<td>1198</td>
<td>1281</td>
</tr>
<tr>
<td>PT bar ( \phi 21 ) (3-RFl column)*</td>
<td>C</td>
<td>346.4</td>
<td>1189</td>
<td>1273</td>
</tr>
</tbody>
</table>

*\( \sigma_y \) of 0.2% offset

<table>
<thead>
<tr>
<th>STEEL</th>
<th>Grade</th>
<th>( A_{\text{normal}} ) (mm(^2))</th>
<th>( F_y ) (kN)</th>
<th>( F_t ) (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>PT wire ( \phi 15.2 ) (ED of wall base)*</td>
<td></td>
<td>140.7</td>
<td>250</td>
<td>277</td>
</tr>
<tr>
<td>PT wire ( \phi 15.2 ) (beam)*</td>
<td></td>
<td>140.7</td>
<td>255</td>
<td>279</td>
</tr>
<tr>
<td>PT wire ( \phi 17.8 ) (beam)*</td>
<td></td>
<td>208.4</td>
<td>356</td>
<td>404</td>
</tr>
<tr>
<td>PT wire ( \phi 19.3 ) (beam)*</td>
<td></td>
<td>243.7</td>
<td>429</td>
<td>481</td>
</tr>
</tbody>
</table>

*\( F_y \) of 0.2% offset

<table>
<thead>
<tr>
<th>CONCRETE</th>
<th>( F_c ) (N/mm(^2))</th>
<th>( \sigma_{\beta} ) (N/mm(^2))</th>
</tr>
</thead>
<tbody>
<tr>
<td>Precast concrete (normal)</td>
<td>60</td>
<td>83.2</td>
</tr>
<tr>
<td>Precast concrete (fiber)</td>
<td>60</td>
<td>85.5</td>
</tr>
<tr>
<td>Top concrete</td>
<td>30</td>
<td>40.9</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>GROUT</th>
<th>( F_c ) (N/mm(^2))</th>
<th>( \sigma_{\beta} ) (N/mm(^2))</th>
</tr>
</thead>
<tbody>
<tr>
<td>Column base, wall base and beam end</td>
<td>60</td>
<td>135.6</td>
</tr>
<tr>
<td>Wall base (fiber)</td>
<td>60</td>
<td>120.3</td>
</tr>
<tr>
<td>PT duct of PT bar and PT wire</td>
<td>30</td>
<td>63.4</td>
</tr>
</tbody>
</table>

The specimen was designed with a typical Japanese PT frame structure in the longitudinal direction, but with a new type of unbonded PT wall-frame structure in the
transverse direction. Table 2.4 lists the reinforcing details. Figure 2.23 shows details of the whole steel arrangement. Beam to column connection detail, details of wall, and the construction procedure are provided in Appendix A.3. The precast concrete members were assembled at the construction site, and then half-precast beams and half-precast slabs were fixed using topping concrete. The half-precast slabs were supported by pretensioned, prestressed beams at 1-m intervals. The design strength of the topping concrete was 30 N/mm². The design strength of the grout mortar was 60 N/mm². The PT reinforcement of the columns was a high-strength steel bar whose nominal strength was 1080 N/mm². The PT reinforcement of beams and walls was high-strength steel strands whose nominal strength was about 1600 N/mm². The PT tendons located in sheaths of columns and beams of the longitudinal direction were grouted. The PT tendons located in sheaths of walls and beams in the transverse direction were not grouted and remained unbonded from anchor to anchor. The normal steel bars cross the wall and foundation interface remained unbonded in half of the first story wall length. The nominal strength of the normal steel bar was 345 N/mm². The column, wall, and beam of the longitudinal direction contained the amount of shear reinforcement required by the Japanese Building Standard Law. In the transverse direction, the walls and beams were confined by high-strength steel bars. The nominal strength of the steel bar was 785 N/mm². In the first and second stories, one of two walls was additionally reinforced by steel fibers.

The corresponding grout beds were reinforced by steel fibers as well. The steel fiber for the wall concrete was 30 mm long with a nominal strength of 1000 N/mm². The steel fiber for grout bed was 10 mm long with a nominal strength of 1500 N/mm². The effective stress of the PT tendon was designed to be 0.6 times of the yield strength for the walls and beams in the exterior frame of the transverse direction. The effective stress of the PT tendon was designed to be 0.8 times of the yield strength for the others.
### Table 2.4 Reinforcement details for PT building.

<table>
<thead>
<tr>
<th>List of Column</th>
<th>PC1</th>
</tr>
</thead>
<tbody>
<tr>
<td>4FL, 3FL, 2FL, 1FL</td>
<td></td>
</tr>
<tr>
<td>Section</td>
<td>450</td>
</tr>
<tr>
<td>Tendon</td>
<td>8-21mm(SBPR1080/1230)</td>
</tr>
<tr>
<td>Rebar</td>
<td>4-D19</td>
</tr>
<tr>
<td>Hoop</td>
<td>2-D10@100</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>List of Girder</th>
<th>PG1</th>
</tr>
</thead>
<tbody>
<tr>
<td>Location</td>
<td>End</td>
</tr>
<tr>
<td>Section</td>
<td>3C-1-15.2mm(SWPR7B)</td>
</tr>
<tr>
<td>Top</td>
<td>2 - D19</td>
</tr>
<tr>
<td>Bottom</td>
<td>3 - D19</td>
</tr>
<tr>
<td>Stirrup</td>
<td>2-D10@150</td>
</tr>
<tr>
<td>Web</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>List of Wall</th>
<th>P Wall</th>
</tr>
</thead>
<tbody>
<tr>
<td>4FL, 3FL, 2FL, 1FL</td>
<td></td>
</tr>
<tr>
<td>Section</td>
<td></td>
</tr>
<tr>
<td>Tendon</td>
<td>3-10-15.2mm(SWPR7B)</td>
</tr>
<tr>
<td>V bar</td>
<td>D13@100(double)</td>
</tr>
<tr>
<td>H bar</td>
<td>D13@100(double)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>List of Slab</th>
<th>Depth: 130mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shorter direction</td>
<td>Longer direction</td>
</tr>
<tr>
<td>PS1</td>
<td>D10@200</td>
</tr>
<tr>
<td>CS1</td>
<td>D13@200</td>
</tr>
<tr>
<td>Bottom</td>
<td>D10@200</td>
</tr>
<tr>
<td>CS2</td>
<td>D10@200</td>
</tr>
<tr>
<td>Bottom</td>
<td>D10@200</td>
</tr>
<tr>
<td>CS3</td>
<td>D13@200</td>
</tr>
<tr>
<td>Bottom</td>
<td>D10@200</td>
</tr>
</tbody>
</table>
In designing the columns and beams in the longitudinal direction, more than 1.5 of the column-to-beam strength ratios was satisfied so that the complete mechanism was based on beam hinges. The strength capacity in the longitudinal direction was set to have the same value as defined in the Japanese Building Standard Law. The PT wall was designed referring to static parametric studies using a fiber model. The study focused primarily on the balance between the amounts of vertical PT tendons and the confinement reinforcements, as well as on the influence to capacity of the normal unbonded steel bars of the base. Basically, the walls satisfied the provisions of ACI ITG-5.2-09. Detailed information of unbonded post-tensioned concrete walls was as follows:

**Unbonded Post-Tensioned Concrete Walls:** The four-story unbonded post-tensioned (UPT) concrete walls were constructed using four precast concrete panels that were
post-tensioned together along horizontal joints. The typical section for the wall panels was 2.5 m long by 250 mm thick with a cross-sectional aspect ratio \( (l_w/t_w) \) of 10. The first, second, and third story wall panels were 3 m high. The fourth story wall panel was extended 450 mm above the roof slab. The extended length of the fourth story wall panel was thickened to 400 mm in order to accommodate anchorage for the post-tensioning reinforcing. The assembled walls had a height-to-length aspect ratio \( (H_w/l_w) \) of 5.

The concrete panels for the North wall were fabricated using a high-performance fiber reinforced cement composite (FRCC). The South wall panels were fabricated using a conventional Portland cement concrete mix with a minimum specified compressive strength of 60 MPa (8.7 ksi). The vertical faces of the panels were reinforced with a two-way mesh of D13 SD295 reinforcing bars. Supplemental D13 SD295 transverse ties were added to prevent separation of the reinforcing mesh from the concrete core, a failure mechanism noted by Perez et al. [2004c]. The mild steel reinforcing was not developed across the panel joints.

The compression zones of the wall panels were reinforced with high-strength S13 KSS785 confinement hoops. In the base wall panel the compression zones were reinforced with two bundled, overlapping S13 KSS785 hoops at a vertical spacing of 75 mm. The confinement reinforcing ratios for the base wall panel, equal to the volumetric ratio of confinement reinforcing to the confined concrete core, were 1.7% for the length-wise direction \( (\rho_x) \), and 1.8% for the thickness direction \( (\rho_y) \). The overall confinement reinforcing ratio \( (\rho_s) \) for the base wall panel was 3.5%. In the upper story panels, the level of confinement was reduced to single S13 KSS785 hoop at 100 mm vertical spacing. The ratio of the total confinement length to the overall length of the wall \( (l_c/l_w) \) was 0.4.

Based on preliminary design results presented at planning meetings at PEER, a wall cross section 250 mm thick and 2500 mm long was selected. According to the AIJ Guidelines, the walls have deformation capacity of more than 2% drift angle for both shear failure and bending compression failure. In the PT Building, the wall was post-tensioned by unbonded strands extending over the full height of the building to provide a mechanism for energy dissipation at the interface of the wall and the foundation. Unbonded reinforcement also was placed across the interface of the wall base and the foundation to provide a mechanism for energy dissipation. The arrangement of the unbonded energy-dissipating reinforcement was
selected based on numerical studies. These studies are briefly described in the following paragraph.

For both the RC and PT Buildings, preliminary analyses were conducted using fiber models to assist with design decisions. Two results are presented for the PT Building, one with two PT strands and no energy-dissipating bars, and the other with two PT strands and 8 energy-dissipating bars (Figure 2.24). Figure 2.25 compares relative strength, hysteretic energy dissipation, and concrete compressive strain for RC and PT walls. The energy dissipation capacity of the PT wall increased four times by providing the unbonded deformed reinforcement at the wall base (and embedded into the foundation). The concrete compressive strain was about four times higher in the PT wall compared with the RC wall. In addition to providing high-strength transverse reinforcement, as was done in the RC wall, steel-fiber reinforced concrete was used over the first two stories of the PT wall.

In order to enhance energy dissipation during seismic response, eight D22 SD345 mild steel reinforcing bars (four at each end) were included across the base panel-foundation interface. The energy-dissipating reinforcing bars were positioned within the central core of the wall (i.e., outside of the compression regions) and were unbonded over a length of 1.5 m within the base wall panel. In order to facilitate construction, the energy-dissipating bars were spliced within the foundation using a grouted coupler.

The post-tensioning in the walls consisted of two bundles of 10-D15.2 SWPR7B post tensioning strands, with a PT steel ratio \( \rho_{pt} \) of 0.44%. The bundled strand groups were positioned symmetrically on either side of the centroidal axis of the wall with an eccentricity of 380 mm. The initial prestress (after release) in the strand groups was equal to 60% of the yield stress for the strand material \( f_{py} \). The corresponding initial compressive stress in the wall due to post-tensioning \( f_{ci,pt} \) was 4.3 MPa (0.62 ksi). Because the bundled strands were contained within ungrouted polyethylene ducts, they were unbonded from the concrete wall panels over the full wall height between mechanical anchorages at the top and bottom of the wall.
Figure 2.24  Hysteretic behavior of cantilever analyses.

Figure 2.25  Strength, hysteresis, energy dissipation, and concrete compressive strain at 2% drift angle.
2.3.1 Design of Unbonded Post-tensioned Concrete Walls

2.3.1.1 Performance-Based Design

Details for the UPT concrete walls were developed using a performance-based design approach. For design purposes, the UPT concrete walls were conservatively analyzed as isolated lateral force resisting components, i.e., the contribution of the light PT frames and the interaction of the walls with the connecting UPT beams and composite floor system were neglected. Two analytical models were developed to characterize the lateral load response of the walls and to estimate design capacities and design demands: (1) an idealized tri-linear lateral load response model; and (2) a rigorous nonlinear finite element model (presented in Section 2.3).

Idealized Tri-Linear Lateral Load Response Model: Previous analytical and experimental studies [Kurama et al. 1996; 1997; 1999a; 1999b; Perez et al. 1998; 2004a; 2004b; 2004c; 2007; Keller and Sause 2010] have demonstrated that the lateral load response of UPT concrete walls can be characterized by the following limit states: (1) decompression (DEC), (2) effective linear limit (ELL), (3) yielding of the post-tensioning steel (LLP), (4) crushing of the confined concrete (CCC), and (5) fracture of the post-tensioning steel (FP). For well-designed and detailed UPT concrete walls, an idealized tri-linear pushover curve (Figure 2.26) can be developed using simplified predictions of response parameters for limit states 2 (ELL), 3 (LLP), and 4 (CCC). Comparisons of response predictions from the idealized tri-linear pushover model with results from previous large-scale experimental tests and detailed nonlinear finite element analyses are presented in Figure 2.27.
Figure 2.26  Idealized tri-linear lateral load response curve for UPT concrete walls [Perez et al. 2004a].

![Idealized tri-linear lateral load response curve](image)

Figure 2.27  Comparison of experimental and analytical results for test wall TW5 [Perez et al. 2004a].

![Comparison of experimental and analytical results](image)

<table>
<thead>
<tr>
<th>Result Type</th>
<th>Loading Direction</th>
<th>DEC</th>
<th>SPL</th>
<th>LLP</th>
<th>CCC</th>
</tr>
</thead>
<tbody>
<tr>
<td>Exp.</td>
<td>Eastward</td>
<td>28.4</td>
<td>0.05</td>
<td>88.9</td>
<td>0.65</td>
</tr>
<tr>
<td></td>
<td>Westward</td>
<td>-29.8</td>
<td>-0.06</td>
<td>-85.9</td>
<td>-0.65</td>
</tr>
<tr>
<td>F.M.A.</td>
<td>Eastward</td>
<td>22.4</td>
<td>0.04</td>
<td>88.1</td>
<td>0.54</td>
</tr>
<tr>
<td></td>
<td>Westward</td>
<td>-22.4</td>
<td>-0.04</td>
<td>-88.2</td>
<td>-0.53</td>
</tr>
<tr>
<td>F.M.A.</td>
<td>Eastward</td>
<td>18.0</td>
<td>0.04</td>
<td>88.9</td>
<td>0.54</td>
</tr>
<tr>
<td></td>
<td>Westward</td>
<td>-18.0</td>
<td>-0.04</td>
<td>-80.6</td>
<td>-0.54</td>
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<tr>
<td>C.F.E.</td>
<td>Eastward</td>
<td>29.6</td>
<td>0.04</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

** CCC was not reached.
LIMIT STATES FOR UPT CONCRETE WALLS:

Decompression (DEC)—Decompression (DEC) occurs when tensile strain demand at the base of the wall, due to overturning moment from lateral loading, equals the pre-compression strain due to post-tensioning and gravity loads. If reinforcing steel is not developed across the horizontal joint at the base of the wall, decompression is accompanied by the initiation of gap opening along the wall base-foundation interface. Under a specified lateral load distribution, decompression of the wall can be related to a specific level of base shear, $V_{dec}$, and roof drift, $\Theta_{dec}$.

Effective Linear Limit (ELL)—The lateral load response of a UPT concrete wall is nearly linear elastic immediately after decompression. As drift levels increase, however, a substantial reduction in lateral stiffness occurs due to nonlinear softening of the concrete in compression and the progression of the gap opening along the horizontal joint at the base of the wall (geometric softening). The lateral stiffness decreases in a smooth and continuous manner, so the term effective linear limit is generally used to describe the point at which softening is apparent. The base shear and roof drift corresponding to the effective linear limit are $V_{ell}$ and $\Theta_{ell}$, respectively.

Yielding of the Post-Tensioning Steel (LLP)—The linear limit for the post-tensioning steel is calculated at the onset of yielding. For simplicity, the axial strain demand is calculated at the centroidal axis of a strand group, i.e., small discrepancies in strain within a group due to the relative eccentricity of the individual strands are neglected. The LLP limit state for the wall is reached when tensile strain demand in the critically stressed group reaches the linear limit for the strand material. The base shear and roof drift corresponding to yielding of the post-tensioning steel are denoted as $V_{lp}$ and $\Theta_{lp}$, respectively.

Crushing of the Confined Concrete (CCC)—Failure of the wall occurs when the confined concrete at the base fails in compression. Based on the confined concrete constitutive model developed by Mander et al. [1988a; 1988b], crushing of the confined concrete occurs at an ultimate concrete compressive strain, $\varepsilon_{cu}$, which is reached when the confinement
reinforcement fractures. Significant loss of lateral load and gravity load resistance are expected to occur when the crushing limit state is reached. The base shear and roof drift corresponding to crushing of the confined concrete are denoted as \( V_{ccc} \) and \( \Theta_{ccc} \), respectively.

**Fracture of the PT Steel (FP)**—Fracture of the PT steel occurs when the tensile strain demand reaches the capacity of the strand material. The limit state is accompanied by a sudden and significant loss of lateral load resistance and self-centering capability. The base shear and roof drift corresponding to fracture of the post-tensioning steel are denoted as \( V_{fp} \) and \( \Theta_{fp} \), respectively.

**DESIGN CRITERIA FOR UPT CONCRETE WALLS**

The following design criteria were developed by Perez et al. [2004c] for UPT concrete walls:

**Criterion 1: Softening**—This design criterion controls softening of the lateral stiffness of the UPT concrete wall for the design level ground motion.

\[
V_{ell} \geq \alpha_d \cdot V_d
\]

where \( V_{ell} \) is the base shear at the effective linear limit, \( \alpha_d \) is a factor applied to the design base shear demand to define the base shear at which softening is allowed to occur (recommended range: 0.65-1.0), and \( V_d \) is the design base shear demand.

**Criterion 2: Base Moment Capacity**—This design criterion controls the base moment capacity of the wall as governed by axial-flexural behavior.

\[
\Phi_f V_{llp} \geq V_d
\]

where \( \Phi_f \) is a capacity reduction factor for flexural strength, and \( V_{llp} \) is the base shear corresponding to the initiation of yielding in the PT steel.

**Criterion 3: Yielding of the Post-Tensioning Steel**—This design criterion controls yielding of the PT steel, which has an adverse effect on drift control and self-centering capability.

\[
\Theta_{llp} \geq \Theta_d
\]
where $\Theta_{lp}$ is the roof drift corresponding to the initiation of yielding in the PT steel and $\Theta_d$ is the roof drift demand for the design level ground motion.

**Criterion 4: Story Drift**—This design criterion controls the maximum story drift for the design level ground motion.

$$\delta_{ali} \geq \delta_{d}$$

where $\delta_{ali}$ is the allowable story drift for the design level ground motion, and $\delta_{d}$ is the story drift demand for the design level ground motion.

**Criterion 5: Crushing of the Confined Concrete**—This design criterion controls the axial-flexural compression failure of the walls.

$$\Theta_{ccc} \geq \Theta_{m}$$

where $\Theta_{ccc}$ is the roof drift corresponding to crushing of the confined concrete, and $\Theta_{m}$ is the roof drift demand for the maximum considered ground motion.

**Criterion 6: Fracture of the Post-Tensioning Steel**—This design criterion ensures that fracture of the PT steel does not occur.

$$\Theta_{fp} \geq \Theta_{ccc}$$

where $\Theta_{fp}$ is the roof drift corresponding to fracture of the PT steel.

**Criterion 7: Roof Drift Limit under the Maximum Considered Ground Motion**—This design criterion limits the drift demand under the maximum considered ground motion to ensure stability of the gravity load system.

$$\Theta_{g} \geq \Theta_{m}$$

where $\Theta_{g}$ is the roof drift corresponding to failure of the gravity load resisting system.
ESTIMATION OF DESIGN CAPACITIES
Preliminary estimates of design capacities for the walls were based on the simplified tri-linear lateral load response model. Perez et al. [2004c] presents simplified expressions for estimating design capacities of UPT concrete walls. Final estimates of design capacities for the walls were based on nonlinear finite element pushover analyses (see Section 2.3).

ESTIMATION OF DESIGN DEMANDS
Design demands for the UPT concrete walls were based on three levels of seismic intensity. Seismic response coefficients ($C_r$) of 0.20 and 0.30 were used to represent the design-basis earthquake (DBE) and the MCE, respectively. In addition, the UPT concrete walls were designed to remain linear elastic up to a seismic response coefficient of 0.15. Preliminary estimates of deformation demands for the UPT concrete walls were estimated using the procedure outlined in Seo and Sause [2005], which accounts for the tangent stiffness of the wall after the effective linear limit (ELL) and hysteretic energy-dissipation characteristics. Nonlinear response history simulations (see Section 2.3) were used to evaluate deformation demands for the proposed test plan.

CONFORMANCE WITH CURRENT U.S. DESIGN PROVISIONS
The UPT concrete wall design satisfies the strength and detailing requirements of ACI ITG-5.2-09 with one notable exception. The PT reinforcing groups are offset from the centroid of the wall by 15% of the wall length. The ACI ITG-5.2-09 was developed for UPT concrete walls with PT reinforcing located within 10% of the wall length from the wall centroid. The experimental program described herein increased the eccentricity of the PT reinforcing steel to 15% to control drift demands, by way of increasing the post-decompression lateral stiffness. The two ground acceleration records selected for the experimental program, from the 1995 Great Hanshin Earthquake produce relatively large spectral acceleration demands in the elongated post-ELL period range of the structure, which significantly increases deformation demands in the structural system.
2.4 CONSTRUCTION

The buildings were constructed between July and October 2010 and moved onto the E-Defense shake table in November 2010. Instrumentation of building was primarily completed in November 2010. The construction process is depicted in Appendix C.

The specimen was constructed outside and then transferred onto the shake table, as shown in Appendix A. The specimen was suspended by two cranes and then set on the shaking table. The foundation beams were strongly fixed by one hundred and fifty post-tensioned PT bars. The foundation beams were constructed on the six concrete stubs, 1.4 m x 3 m x 1.5 m in configuration, to leave enough space for the carrier access beneath the specimen. The foundation beams were 1200 mm deep and designed for each phase of the test program, from the construction to set up, by using the supplementary PT tendons to prevent excessive cracks. The concrete was cast for the columns, walls, upper floor beams, and the floor slab. The main reinforcement of columns, beams, and the assumed column-zones of walls were connected by gas pressure welding. Lap joints were used for reinforcing the walls and floor slabs.
3 Test Plan and Instrumentation

The two test buildings were heavily instrumented to assess their performance when subjected to a range of shaking intensities for a range of post-test analytical studies. The table motions used for the testing and the instrumentation used for each of the two buildings are briefly described in the following sections. Additional information is provided in Appendix D.

3.1 TEST PLAN

The 1995 JMA-Kobe and JR-Takatori records were selected for this experimental program. Testing was conducted on December 13th and December 15th, subjecting the buildings to the JMA-Kobe record, and a third test was conducted using the JR-Takatori record on December 17th. The NS-direction acceleration, EW-direction acceleration, and vertical-direction acceleration were aligned with the transverse-direction (y), longitudinal direction (x), and vertical direction of the specimen (Figure 2.3). Natural periods 0.36 and 0.18 were computed for the models (see Chapter 2) for the shear wall (y) and moment frame (x) directions, respectively. In the tests the amplitude associated with the JMA-Kobe record was scaled to produce a range of shaking intensities; scale factors of 25 %, 50 %, and 100 % were used. The orbit of horizontal acceleration is shown in Figures 3.9-3.10. Based on preliminary analyses, the stronger NS-direction wave was input into the transverse-direction. The two tests run with the JR-Takatori record were scaled to 40% and 60%.
3.2 INSTRUMENTATION

3.2.1 General

A total of 609 channels of data were collected during the tests for RC and PT specimens, including 48 accelerometers, 202 displacement transducers, and 235 strain gauges. The accelerometers were placed on the foundation and on each floor slab to record accelerations in three directions. Displacement transducers were arranged to measure interstory displacements, beam end rotations, column end rotations, and base wall rotations. Strain gauges were glued to longitudinal and transverse reinforcement of beams, columns, and walls. Strain gauges were largely used for the RC specimen, whereas displacement transducers were used for the PT specimen (to measure member end rotations). Video cameras were used to record the tests and included overall views of the test specimens, as well as close up views of regions where yielding and damage were anticipated. Data acquisition was accomplished using 24 bit A/D converters using a sample rate of 0.001 sec (1000 Hz). Locations of instrumentation are shown in Appendix D.

3.2.2 Types of instrumentation

Figure 3.1 shows properties of the three different types of instrumentation that were used for the tests: accelerometers, displacement transducers, and strain gauges.

3.2.2.1 Accelerometers

Accelerometers were used to record accelerations at each floor. Figure 3.2 shows the locations of accelerometers. Detailed information is provided in Appendix D.
Figure 3.1  Properties of the instrumentation used in the specimens.
3.2.2.2 Displacement Transducers

A total of 202 displacement transducers were used for the tests, including wire potentiometers, laser-type displacement transducers, and linear variable differential transducers (LVDTs). The transducers were attached to the test specimens to measure horizontal and vertical displacements, lateral story displacements and drifts, average concrete strains over gauge lengths, pullout/gapping at member ends, and sliding at the base of the shear walls. Locations of wire and laser transducers are shown in Figures 3.3 and 3.4.

A majority of the LVDTs were provided by NIED; however, some of the displacements transducers were provided by NEES@UCLA, IOWA State University, and the Earthquake Research Institute at the University of Tokyo; this enabled more detailed measurements of wall deformations (Figures 3.5 and 3.6). Four transducers were used over a gauge length of 540 mm at the base of the walls to enable the curvature along the wall length (depth) to be determined (Figure 3.5); additional displacement transducers were provided at each wall boundary over the entire height of the building (Figure 3.5). Two pairs of diagonally-oriented displacement transducers were used over the first story height to enable
the determination of shear deformations. Photographs showing the displacement transducers over the first story height of the RC building are shown in Figure 3.7. Further information is provided in Appendix D.

**Strain Gauges:** Reinforcement strains were measured at 235 locations using strain gauges. Figure 3.8 shows the locations of the strain gauges in horizontal and vertical reinforcement in RC building at the first and second floor. More detailed information is provided in Appendix D.

![Figure 3.3 Locations of the wire-type displacement transducers.](image-url)
Figure 3.4  Locations of the laser-type displacement transducers.

Figure 3.5  Vertical LVDT configuration (first floor).
Figure 3.6  Diagonal LVDT configuration (first floor).

Figure 3.7  Instrumentation on the RC wall.
Figure 3.8 Strain gauge locations in horizontal and vertical directions at the first floor (RC).
3.3 GROUND MOTIONS

Two different table motions at various intensities were used: JMA-Kobe (25%, 50%, and 100%) and Takatori (40% and 60%). The testing was planned over five days: low-to-moderate intensity JMA-Kobe (25% and 50%) on December 13, 2010, 100% JMA-Kobe on December 15, 2010, and Takatori (40% and 60%) on December 17, 2010.

Pseudo acceleration spectra of the JMA-Kobe ground motions are presented in Figures 3.9 and 3.10 for the $x$- (frame) direction and $y$- (shear wall) directions, respectively. The broken lines show the target spectrum, whereas solid lines illustrate the actual spectra determined from measurements. Peak spectral accelerations observed on the shaking table were 0.58g at 25%, 1.18g at 50% and 2.79g at 100% JMA-Kobe in the frame direction; and 0.89g at 25%, 1.58g at 50% and 3.42g at 100% JMA-Kobe in the shear wall direction.

Pseudo acceleration spectra of the Takatori ground motions were also plotted (see Figures 3.11 and 3.12). At 40%, the Takatori record had a peak spectral acceleration of 1.11g and 0.99g in the frame and shear wall directions, respectively. At 60%, the Takatori record had a peak spectral acceleration of 1.72g in the frame direction and 1.51g in the shear wall directions, respectively.

Displacement spectra are shown in Figures 3.13-3.16. Peak spectral displacements were observed as 10.5 cm at 25%, 20.9 cm at 50%, and 41.8 cm at 100% JMA-Kobe; and 40.3 cm at 40%, and 60.2 cm at the 60% Takatori in the frame direction. In the other direction, the peak displacements were 11.6 cm at 25%, 23 cm at 50%, and 46 cm at the 100% JMA-Kobe record; and 48.1 cm at the 40%, and 72.3 cm at the 60% Takatori records.
Figure 3.9 Acceleration spectra for JMA-Kobe ground motion (x-direction).

Figure 3.10 Acceleration spectra for JMA-Kobe ground motion (y-direction).
Figure 3.11 Acceleration spectra for Takatori ground motion (x-direction).

Figure 3.12 Acceleration spectra for Takatori ground motion (y-direction).
Figure 3.13 Displacement spectra for the Kobe ground motion (x-direction).

Figure 3.14 Displacement spectra for the Kobe ground motion (y-direction).
Figure 3.15  Displacement spectra for the Takatori ground motion (x-direction).

Figure 3.16  Displacement spectra for the Takatori ground motion (y-direction).
4 Summary, Conclusions, and Future Work

4.1 SUMMARY

Detailed information related to the December 2010 tests of two, full-scale, four-story buildings that were tested on the NIED E-Defense shake table are presented. Substantial collaboration between U.S. and Japan researchers over a period of nearly two years preceded the shake table testing. The goal of the collaboration was to produce test buildings that would provide vital data on behavior and response over a spectrum on shaking intensities, including near-collapse, for research efforts in both the U.S. and Japan.

The tests were successfully completed during the week of December 13-17, 2010. The large number of instruments placed, including video cameras, will provide a wealth of data that will enable both Japanese and U.S. researchers to improve our understanding of the behavior of these systems. Papers that summarize the overall results are being prepared for submittal to AIJ and a U.S. journal by mid-summer 2011.

Support has been provided by NEEScomm to conduct a blind prediction study associated with the RC and PT Building tests. The data in this report are intended to provide background information to support this effort.

4.2 FUTURE STUDIES

A subsequent report will be prepared that provides an overview of the test results and pre-test analytical studies, as well as post-test studies.
REFERENCES


Appendix A

A.1 MATERIAL PROPERTIES

Actual material properties for RC specimen

<table>
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<tr>
<th>Steel</th>
<th>Grade</th>
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<th>$\sigma_y$ (N/mm$^2$)</th>
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*$\sigma_y$ of 0.2% offset (shear reinforcement)

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<td>3rd - 4th floor</td>
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<td>4th - roof floor</td>
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### Actual material properties for PT specimen

#### Steel

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<td>563</td>
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* $\sigma_y$ of 0.2% offset

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<td>PT wire $\phi$15.2 (beam)*</td>
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<td>PT wire $\phi$17.8 (beam)*</td>
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<td>PT wire $\phi$19.3 (beam)*</td>
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* $F_y$ of 0.2% offset

#### Concrete

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A.2 MEMBER GEOMETRY AND REINFORCEMENT OF THE RC SPECIMEN

Figure A.1  Floor plan of the RC specimen.

Figure A.2  Elevation of the RC specimen.
Figure A.3  Overview of the RC specimen.
Table A.1 List of steel reinforcement

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Figure A.4  Details of RC specimen.
Figure A.5  Steel locations at floor 1F.
Figure A.6  Steel locations at floor 2F.
Figure A.8  Steel locations at floor 3F.
Figure A.9  Steel locations at floor 3F.
Figure A.10  Steel locations at floor 4F.
Figure A.11  Steel locations at floor 4F.
A.2 MEMBER GEOMETRY AND REINFORCEMENT OF THE PT SPECIMEN

Figure A.12  Floor plan of the PT specimen.

Figure A.13  Elevation of the PT specimen.
Figure A.14  Overview of the PT specimen.
Table A.2 List of steel reinforcement.

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<tr>
<td>Tendon</td>
<td>3-10-15.2mm(SWPR7B)</td>
</tr>
<tr>
<td>V bar</td>
<td>D13@150(double)</td>
</tr>
<tr>
<td>H bar</td>
<td>D13@100(double)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>List of Girder</th>
<th>PG2</th>
<th>PG3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Location</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Top</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bottom</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>List of Slab</th>
<th>Depth: 130mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>CS1</td>
<td>D10@200 D10@200</td>
</tr>
<tr>
<td>CS2</td>
<td>Top D10@200 D10@200 Bottom D10@250</td>
</tr>
<tr>
<td>CS3</td>
<td>Top D10@200 D10@200 Bottom D10@200</td>
</tr>
</tbody>
</table>
Figure A.15  Details of PT specimen.
Figure A.16 Details of PT beam column joint.
Figure A.17  Details of PT wall base and foundation.
Figure A.18  Details of PT wall floor slab interface
A.3 SETUP AND PLACEMENT OF THE SPECIMENS

Figure A.19  Set up of the specimens.
Figure A.20 Placement of the specimens on the shaking table.
The specimens can be weighted when they are carried on the shaking table.
Photo 3. RC Specimen Hanging on Cranes: $(248 + 365 \times 25 = 588 \text{ t})$
(Total weight of hanging wires is 25 t)

Photo 4. Weight of PT Specimen Hanging on Cranes: $335 + 236 \times 25 = 546 \text{ t}$
(Total weight of hanging wires is 25 t)

<table>
<thead>
<tr>
<th></th>
<th>Measured</th>
<th>Estimated (Vol. \times 2.4 [t/m$^3$] + Machines)</th>
<th>Ratio (Estimated/Measured)</th>
</tr>
</thead>
<tbody>
<tr>
<td>RC</td>
<td>588 t</td>
<td>595.9 t</td>
<td>101.3 %</td>
</tr>
<tr>
<td>PT</td>
<td>546 t</td>
<td>558.8 t</td>
<td>102.3 %</td>
</tr>
</tbody>
</table>

Figure A.21 Measuring weight of the specimens.
Figure A.22  Weights of equipment on the buildings at the third level
Figure A.23  Weights of equipment on the buildings at roof level.
Appendix B

B.1 EQUIVALENT LATERAL LOAD PROCEDURE (ASCE 7-05)

SHEAR WALL DIRECTION

Mapped MCE spectral response accelerations:

\[ S_s (g) = 1.5 \] At short periods
\[ S_1 (g) = 0.9 \] At 1 s.

Site coefficients:

\[ F_a = 1 \]
\[ F_v = 1 \]

Importance factor:

\[ I = 1 \]

Response modification factor:

\[ R = 6 \]

Story height:

\[ h_i = 3 \text{ m } 9.84 \text{ ft} \]

Number of stories:

\[ n = 4 \]

Design spectral response acceleration parameters:

\[ S_{MS} (g) = 1.5 \]
\[ S_{M1} (g) = 0.9 \]
\[ S_{DS} (g) = 1 \]
\[ S_{D1} (g) = 0.6 \]
\[ T_s (\text{sec}) = 0.6 \]
\[ T_0 (\text{sec}) = 0.12 \]

Period Calculations:

Eigenvalue analysis:

\[ T_{eigen} = 0.58 \text{ sec} \]

Approximate period:

Table 12.8-2:

\[ C_t = 0.0488 \] (for metric)
\[ h_n = 12 \text{ m} \]
\[ x = 0.75 \]

ASCE 7-05 (12.8): \[ C_t^* (h_n)^x \]
\[ T_a (\text{sec}) = 0.315 \]

ASCE 7-05 (12.8): 0.1N
\[ T_a (\text{sec}) = 0.4 \]
\[
C_u = 1.4 \quad (S_{D1} > 0.6) \\
T_{\text{limit}} = C_u T_a = 0.44
\]

\[T (\text{sec}) = 0.440\]

**Seismic Response Coefficient:**

<table>
<thead>
<tr>
<th>(C_s) (12.8-2)</th>
<th>(C_{s_{\text{max}}}) (12.8-3)</th>
<th>(C_{s_{\text{min}}}) (12.8-5)</th>
<th>(C_{s_{\text{min}}}) (12.8-6)</th>
<th>Weight (kN)</th>
<th>(V_{\text{base}} = C_s \times W)</th>
<th>k</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.167</td>
<td>0.23</td>
<td>0.01</td>
<td>0.075</td>
<td>1815</td>
<td>302.50</td>
<td>1</td>
</tr>
</tbody>
</table>

**Story forces:**

<table>
<thead>
<tr>
<th>i</th>
<th>(w_i) (kN)</th>
<th>(C_{vi})</th>
<th>(F_i) (kN)</th>
<th>(M_i) (kN-m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>441</td>
<td>0.096</td>
<td>29.1</td>
<td>2745</td>
</tr>
<tr>
<td>2</td>
<td>444.5</td>
<td>0.194</td>
<td>58.6</td>
<td>1838</td>
</tr>
<tr>
<td>3</td>
<td>458</td>
<td>0.299</td>
<td>90.6</td>
<td>1017</td>
</tr>
<tr>
<td>4</td>
<td>471.5</td>
<td>0.411</td>
<td>124.3</td>
<td>373</td>
</tr>
</tbody>
</table>

Total weight: 1815

\[V_{\text{base}} = 302.5\]

**Weight is half the full weight to find the forces per shear wall system.**

**Redundancy Factor \(\rho\) = 1.3**

**Story forces with redundancy factor**

\[E_h = \rho Q_e\]

<table>
<thead>
<tr>
<th>(i)</th>
<th>(F_i) (kN)</th>
<th>(M_i) (kN-m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>37.8</td>
<td>3569</td>
</tr>
<tr>
<td>2</td>
<td>76.2</td>
<td>2389</td>
</tr>
<tr>
<td>3</td>
<td>117.7</td>
<td>1323</td>
</tr>
<tr>
<td>4</td>
<td>161.6</td>
<td>485</td>
</tr>
</tbody>
</table>

\[E_h = V \times \rho = 393.3\quad \text{kN}\]

\% of weight = 21.67 %
FRAME DIRECTION

Mapped MCE spectral response accelerations:

\[ S_s (g) = 1.5 \quad \text{At short periods} \]
\[ S_1 (g) = 0.9 \quad \text{At 1 s.} \]

Site coefficients:

\[ F_a = 1 \]
\[ F_v = 1 \]

Importance factor:
\[ I = 1 \]

Response modification factor:
\[ R = 8 \]

Story height:
\[ h_i = 3 \quad \text{m} \quad 9.84 \quad \text{ft} \]

Number of stories:
\[ n = 4 \]

Design spectral response acceleration parameters:

\[ S_{MS} (g) = 1.5 \]
\[ S_{M1} (g) = 0.9 \]
\[ S_{DS} (g) = 1 \]
\[ S_{D1} (g) = 0.6 \]
\[ T_s \quad (\text{sec}) = 0.6 \]
\[ T_0 \quad (\text{sec}) = 0.12 \]

Period Calculations:

Eigenvalue analysis:
\[ T_{\text{eigen}} = 0.67 \quad \text{sec} \]

Approximate period:

Table 12.8-2:
\[ C_t = 0.0466 \quad \text{(for metric)} \]
\[ h_n = 12 \quad \text{m} \]
\[ x = 0.9 \]

ASCE 7-05 (12.8):
\[ C_t^* (h_n)^x \quad T_a \quad (\text{sec}) = 0.44 \]

ASCE 7-05 (12.8):
\[ 0.1N \quad T_a \quad (\text{sec}) = 0.4 \]
\[ C_u = 1.4 \quad (S_{D1}>0.6) \]
\[ T_{\text{limit}} = \]
\[ C_u T_a = 0.56 \]
\[ T \quad (\text{sec}) = 0.56 \]

Seismic response Coefficient:

<table>
<thead>
<tr>
<th>( C_s ) (12.8-2)</th>
<th>( C_{s_{\text{max}}} ) (12.8-3)</th>
<th>( C_{s_{\text{min}}} ) (12.8-5)</th>
<th>( C_{s_{\text{min}}} ) (12.8-6)</th>
<th>Weight (kN)</th>
<th>( V_{\text{base}} ) = ( C_s \times W ) k</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.125</td>
<td>0.13</td>
<td>0.01</td>
<td>0.05625</td>
<td>1815</td>
<td>226.88</td>
</tr>
</tbody>
</table>
Story forces:

<table>
<thead>
<tr>
<th>i</th>
<th>wi (kN)</th>
<th>Cvi</th>
<th>Fi (kN)</th>
<th>Mi(kN-m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>441</td>
<td>0.093</td>
<td>21.1</td>
<td>2067</td>
</tr>
<tr>
<td>2</td>
<td>444.5</td>
<td>0.192</td>
<td>43.5</td>
<td>1387</td>
</tr>
<tr>
<td>3</td>
<td>458</td>
<td>0.300</td>
<td>68.0</td>
<td>769</td>
</tr>
<tr>
<td>4</td>
<td>471.5</td>
<td>0.415</td>
<td>94.2</td>
<td>283</td>
</tr>
</tbody>
</table>

Total weight: 1815

\[ V_{\text{base}} = 226.9 \]

** Weight is half the full weight to find the forces per special moment frame.

Redundancy Factor \( \rho = 1.3 \)

Story forces with redundancy factor

\[ E_h = \rho Q_{\varepsilon} \]

<table>
<thead>
<tr>
<th>i</th>
<th>Fi (kN)</th>
<th>Mi(kN-m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>27.5</td>
<td>2687</td>
</tr>
<tr>
<td>2</td>
<td>56.5</td>
<td>1803</td>
</tr>
<tr>
<td>3</td>
<td>88.5</td>
<td>1000</td>
</tr>
<tr>
<td>4</td>
<td>122.5</td>
<td>367</td>
</tr>
</tbody>
</table>

\[ E_h = V^*\rho = 294.9 \text{ kN} \]

% of weight = 16.25 %
B.2 CALCULATIONS BASED ON ACI 318-08 PROVISIONS

BEAMS

Materials

Concrete:

\[ f'_c = 27 \text{ MPa} \quad 3.9 \text{ ksi} \]

Steel:

\[ f_y = 345 \text{ MPa} \quad 50.0 \text{ ksi} \]

2G1 (Frame direction)

Cross-section

\[
\begin{align*}
hi &= 3000 \text{ mm} \quad 118.11 \text{ in} \\
Total \ height &= 12000 \text{ mm} \quad 472.44 \text{ in} \\
bw &= 300 \text{ mm} \quad 11.81 \text{ in} \\
h &= 600 \text{ mm} \quad 23.62 \text{ in} \\
Ag &= 180000 \text{ mm}^2 \quad 279.00 \text{ in}^2 \\
Diahoop &= 10 \text{ mm} \quad 0.39 \text{ in} \\
Ahoop &= 78.54 \text{ mm}^2 \quad 0.12 \text{ in}^2 \\
spacings &= 200 \text{ mm} \quad 7.87 \text{ in} \\
d &= 545.5 \text{ mm} \quad 21.48 \text{ in} \\
DiaBar &= 22 \text{ mm} \quad 0.87 \text{ in} \\
Abar &= 380.13 \text{ mm}^2 \quad 0.59 \text{ in}^2 \\
DiaWeb &= 10 \text{ mm} \quad 0.39 \text{ in} \\
AWeb &= 78.54 \text{ mm}^2 \quad 0.12 \text{ in}^2 \\
slab \ thickness &= 130 \text{ mm} \quad 5.12 \text{ in} \\
slab \ reinforcement &= \text{D10 @ 250} \\
\end{align*}
\]

Strength check:

Flexural strength

effective beam width:

\[ ln = 6700 \text{ mm} \quad 263.78 \text{ in} \]

S.8.12.2 : \( beff = \min(ln/4, bw+2*ts, bw+2*[1/2\text{clear dist. to the next web}]) \)

\[
\text{clear dist to the next web} = 6900 \text{ mm} \quad 271.65 \text{ in}
\]
overhanging = 1400 mm 55.12 in

Max total length from:
1) Total length <= ln/4 beff= 1675 mm 65.94 in
2) Each side <= 8ts beff= 2380 mm 93.70 in
3) Each side <= lc/2 beff= 7200 mm 283.46 in

beff= 1675 mm 65.94 in

\[ M_n^+ = 385.70 \text{ kN-m} \]
\[ M_n^- = 572.20 \text{ kN-m} \]
\[ M_{n,center} = 424.60 \text{ kN-m} \]
\[ M_{n,\text{max}} = 572.20 \text{ kN-m} \]

**S21.5.2.2**
\[
\begin{array}{ccc}
\hline
\text{mid-span} & M_n^+ & 385.70 \\
\text{          } & > & M_n^-/2 = 286 \text{ OK} \\
\hline
\end{array}
\]

**S21.5.2.1**
\[
\begin{array}{ccc}
\hline
\text{As,\text{min}} = 3\sqrt{f'c}/fy*bw*d \\
\text{not less than} 200bw*d/fy \\
\hline
\end{array}
\]

As,\text{min} = 614.07 mm\text{\(^2\)} 0.95 in\text{\(^2\)}
200bw*d/fy = 654.27 mm\text{\(^2\)} 1.01 in\text{\(^3\)}

Try 3,6
\[
\begin{array}{ccc}
\hline
\text{# of bars} = 3 \\
\text{current As} = 1140.40 \text{ mm}\text{\(^2\)} 1.77 in\text{\(^3\)} \\
\hline
\end{array}
\]
current As = 1140.40 > As,\text{min} = 654 \text{ OK}

check reinf. Ratio \[
\rho_t = 0.0070
\]
\[
\rho_t = 0.0070 < \rho_{t,\text{max}} = 0.0250 \text{ OK}
\]

**Shear strength**

\[
V_c = 2*\sqrt{f'c}\cdot bw\cdot d
\]

\[
V_c = 141.19 \text{ kN} 31.74 \text{ kips}
\]

\[
\text{# of hoops} = 2
\]

\[
A_v = 157.08 \text{ mm}\text{\(^2\)} 0.24 \text{ in}\text{\(^3\)}
\]

\[
V_s = A_v\cdot fy\cdot d/s
\]

Vs = 147.76 kN 33.22 kips
\[ V_n = V_c + V_s = 288.96 \text{ kN} \quad 64.96 \text{ kips} \]

\[ Vu,pr \text{ due to moments} \]

\[ M_{n,pr}^+ = 482.13 \text{ kN-m} \]
\[ M_{n,pr}^- = 715.25 \text{ kN-m} \]

\[ w_g = 10.40 \text{ N/mm} \]

\[ V_n = 288.96 \text{ kN} \quad 64.96 \text{ kips} \]
\[ Vu,pr = 214 \text{ kN} \quad 48.01 \text{ kips} \]

\[ \Phi V_n = 217 \quad > \quad Vu = 214 \quad \text{OK} \]

**Detailing:**

**Transverse reinforcement**

S21.5.3.1: hoops shall be provided in 2h

\[ 2h = 1200 \text{ mm} \quad 47.24 \text{ in} \]

\[ \text{current region length} = - \text{ mm} \quad - \text{ in} \]

S21.5.3.2: max spacing in 2h:

\[ s = \min(d/4; 8d_b; 24d_{hoop}; 12") = 136.38 \text{ mm} \quad 5.37 \text{ in} \]

\[ \text{current spacing} = 200 \text{ mm} \quad 7.87 \text{ in} \]

\[ \text{current spacing} = 200 \quad > \quad s,\text{min} = 136 \quad \text{NOT OK} \]

**beyond 2h:**

\[ s = d/2 = 272.75 \text{ mm} \quad 10.74 \text{ in} \]

\[ \text{current spacing} = 200 \text{ mm} \quad 7.87 \text{ in} \]

\[ \text{current spacing} = 200 \quad < \quad s,\text{min} = 273 \quad \text{OK} \]

**3G1 (Frame direction)**

**Cross-section**

\[ hi = 3000 \text{ mm} \quad 118.11 \text{ in} \]
\[ \text{Total height} = 12000 \text{ mm} \quad 472.44 \text{ in} \]
\[ bw = 300 \text{ mm} \quad 11.81 \text{ in} \]
\[ h = 600 \text{ mm} \quad 23.62 \text{ in} \]
Ag = 180000 mm² 279.00 in²
Diahoop= 10 mm 0.39 in
Ahoop= 78.54 mm² 0.12 in²
spacing= 200 mm 7.87 in
d= 545.5 mm 21.48 in
DiaBar = 22 mm 0.87 in
Abar= 380.13 mm² 0.59 in²
DiaWeb = 10 mm 0.39 in
AWeb= 78.54 mm² 0.12 in²

slab thickness ts= 130 mm 5.12 in
slab reinforcement D10 @ 250

Strength check:

Flexural strength
effective beam width:

ln= 6700 mm 263.78 in

S.8.12.2 : beff=min(ln/4,bw+2*[8ts],bw+2*[1/2(clear dist. to the next web)])

clear dist to the next web = 6900 mm 271.65 in
overhanging = 1400 mm 55.12 in

Max total length from:
1) Total length <= ln/4 beff= 1675 mm 65.94 in
2) Each side <= 8ts beff= 2380 mm 93.70 in
3) Each side <= lc/2 beff= 7200 mm 283.46 in

beff= 1675 mm 65.94 in

Mn⁺ = 380.30 kN-m
Mn⁻ = 526.90 kN-m
Mn,center= 424.60 kN-m
Mn max = 526.90 kN-m

S21.5.2.2 Mn⁺ = 380.30 > Mn⁻/2= 263 OK
mid-span Mn⁺ = 380.30 > Mn max/4 = 132 OK
Mn⁻ = 526.90 > Mn max/4 = 132 OK

S21.5.2.1 As,min = 3sqrt(f'c)/fy*bw*d
not less than 200bw*d/fy

\[ As_{\text{min}} = 614.07 \text{ mm}^2 \quad 0.95 \text{ in}^2 \]
\[ 200bw*d/fy = 654.27 \text{ mm}^2 \quad 1.01 \text{ in}^3 \]

Try 3,5

\[ \# \text{ of bars} = 3 \]
\[ \text{current } As = 1140.40 \text{ mm}^2 \quad 1.77 \text{ in}^3 \]

current As = 1140.40 > As, min = 654 [OK]

check reinf. Ratio

\[ pt = 0.0070 \]

\[ pt = 0.0070 \quad < \quad pt, \text{ max} = 0.0250 \quad \text{OK} \]

**Shear strength**

\[ V_c = 2 \sqrt{f'c} \times bw \times d \]
\[ V_c = 141.19 \text{ kN} \quad 31.74 \text{ kips} \]
\[ \text{# of hoops} = 2 \]
\[ Av = 157.08 \text{ mm}^2 \quad 0.24 \text{ in}^3 \]
\[ Vs = Av \times fy \times d / s \]
\[ Vs = 147.76 \text{ kN} \quad 33.22 \text{ kips} \]

\[ V_n = V_c + Vs = 288.96 \text{ kN} \quad 64.96 \text{ kips} \]

**Vu,pr due to moments**

\[ M_{n,pr} = 475.38 \text{ kN-m} \]
\[ M_{n,pr} = 658.63 \text{ kN-m} \]
\[ wg = 10.40 \text{ N/mm} \]
\[ V_n = 288.96 \text{ kN} \quad 64.96 \text{ kips} \]
\[ Vu_{,pr} = 204 \text{ kN} \quad 45.88 \text{ kips} \]

\[ \Phi V_n = 217 \quad > \quad Vu = 204 \quad \text{OK} \]

**Detailing:**

**Transverse reinforcement**

S21.5.3.1: hoops shall be provided in 2h

\[ 2h = 1200 \text{ mm} \quad 47.24 \text{ in} \]
current region length = - mm - in

S21.5.3.2: max spacing in 2h:

\[
s = \min \left( \frac{d}{4}; 8d_b; 24d_{\text{hoop}}; 12" \right) = 136.38 \text{ mm} \quad 5.37 \text{ in}
\]

current spacing= 200

\[
s, \text{min} = 136 \quad \text{NOT OK}
\]

beyond 2h:

\[
s \leq \frac{d}{2} = 272.75 \text{ mm} \quad 10.74 \text{ in}
\]

current spacing= 200

\[
s, \text{min} = 273 \quad \text{OK}
\]

**4G1,RG1 (Frame direction)**

**Cross-section**

\[
\begin{align*}
hi & = 3000 \text{ mm} \quad 118.11 \text{ in} \\
\text{Total height} & = 12000 \text{ mm} \quad 472.44 \text{ in} \\
bw & = 300 \text{ mm} \quad 11.81 \text{ in} \\
h & = 600 \text{ mm} \quad 23.62 \text{ in} \\
Ag & = 180000 \text{ mm}^2 \quad 279.00 \text{ in}^2 \\
Diahoop & = 10 \text{ mm} \quad 0.39 \text{ in} \\
Ahoop & = 78.54 \text{ mm}^2 \quad 0.12 \text{ in}^2 \\
\text{spacing} & = 200 \text{ mm} \quad 7.87 \text{ in} \\
d & = 545.5 \text{ mm} \quad 21.48 \text{ in} \\
DiaBar & = 22 \text{ mm} \quad 0.87 \text{ in} \\
Abar & = 380.13 \text{ mm}^2 \quad 0.59 \text{ in}^2 \\
DiaWeb & = 10 \text{ mm} \quad 0.39 \text{ in} \\
AWeb & = 78.54 \text{ mm}^2 \quad 0.12 \text{ in}^2
\end{align*}
\]

slab thickness

\[
ts = 130 \text{ mm} \quad 5.12 \text{ in}
\]

slab reinforcement

\[
\text{D10 @ 250}
\]

**Strength check:**

**Flexural strength**

**effective beam width:**

\[
\ln = 6700 \text{ mm} \quad 263.78 \text{ in}
\]

S.8.12.2 : beff=\(\min(ln/4,bw+2*[8ts],bw+2*[1/2(\text{clear dist. to the next web})])\)
clear dist to the next web = 6900 mm 271.65 in
overhanging = 1400 mm 55.12 in

Max total length from:

1) Total length <= ln/4
   beff= 1675 mm 65.94 in
2) Each side <= 8ts
   beff= 2380 mm 93.70 in
3) Each side <= lc/2
   beff= 7200 mm 283.46 in

\[ \text{beff} = 1675 \text{ mm} \]

\[ M_n^+ = 372.80 \text{ kN-m} \]
\[ M_n^- = 475.40 \text{ kN-m} \]
\[ M_{n,center} = 424.60 \text{ kN-m} \]
\[ M_{n,max} = 475.40 \text{ kN-m} \]

S21.5.2.2

\[ M_n^+ = 372.80 > M_n^- /2 = 238 \text{ OK} \]

mid-span

\[ M_n^+ = 372.80 > M_{n,max} /4 = 119 \text{ OK} \]
\[ M_n^- = 475.40 > M_{n,max} /4 = 119 \text{ OK} \]

S21.5.2.1

\[ A_s,\text{min} = 3\sqrt{f'c}/fy*bw*d \]
not less than \( 200bw*d/fy \)

\[ A_s,\text{min} = 614.07 \text{ mm}^2 \]
\[ 200bw*d/fy = 654.27 \text{ mm}^2 \]

Try 3,4

\# of bars = 3

\[ \text{current } A_s = 1140.40 \text{ mm}^2 \]

\[ \text{current } A_s = 1140.40 > A_s,\text{min} = 654 \text{ OK} \]

check reinf. Ratio

\[ \rho_t = \frac{0.0070}{0.0070} < \rho_{t,\text{max}} = 0.0250 \text{ OK} \]

Shear strength

\[ V_c = 2*\sqrt{f'c}*bw*d \]

\[ V_c = 141.19 \text{ kN} \]
\[ 31.74 \text{ kips} \]

\# of hoops = 2

\[ A_v = 157.08 \text{ mm}^2 \]
\[ 0.24 \text{ in}^3 \]

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\( V_s = A v * f_y * d/s \)  
\[ V_s = 147.76 \text{ kN} \quad 33.22 \text{ kips} \]

\( V_n = \)  
\[ V_c + V_s = 288.96 \text{ kN} \quad 64.96 \text{ kips} \]

**Vu,pr due to moments**

\( M_{n,pr}^+ = 466.00 \text{ kN-m} \)
\( M_{n,pr}^- = 594.25 \text{ kN-m} \)

\( w_{g} = 10.40 \text{ N/mm} \)

\[ V_n = 288.96 \text{ kN} \quad 64.96 \text{ kips} \]
\[ V_{u,pr} = 193 \text{ kN} \quad 43.41 \text{ kips} \]

\[ \Phi V_n = 217 > \quad Vu = 193 \quad \boxed{\text{OK}} \]

**Detailing:**

**Transverse reinforcement**

S21.5.3.1: hoops shall be provided in 2h

\[ 2h = 1200 \text{ mm} \quad 47.24 \text{ in} \]

current region length = - mm - in

S21.5.3.2: max spacing in 2h:

\[ s = \min(d/4; 8d_b; 24d_{hoop}; 12") = 136.38 \text{ mm} \quad 5.37 \text{ in} \]

current spacing= 200 mm 7.87 in

current spacing= 200 > s,min = 136 \boxed{\text{NOT OK}}

**beyond 2h:**

\[ s\leq d/2 = 272.75 \text{ mm} \quad 10.74 \text{ in} \]

current spacing= 200 mm 7.87 in

current spacing= 200 < s,min = 273 \boxed{\text{OK}}

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CORNER COLUMNS

Materials

Concrete:  \( f'_c = 27 \text{ MPa} \quad 3.9 \text{ ksi} \)
Steel:  \( f_y = 345 \text{ MPa} \quad 50.0 \text{ ksi} \)

1C1 (Frame Direction) -- Corner Column

Cross-section

<table>
<thead>
<tr>
<th>Item</th>
<th>Value</th>
<th>Conversion</th>
</tr>
</thead>
<tbody>
<tr>
<td>( h_i )</td>
<td>300 mm</td>
<td>118.11 in</td>
</tr>
<tr>
<td>Total height</td>
<td>12000 mm</td>
<td>472.44 in</td>
</tr>
<tr>
<td>( H_c )</td>
<td>500 mm</td>
<td>19.69 in</td>
</tr>
<tr>
<td>( B_c )</td>
<td>500 mm</td>
<td>19.69 in</td>
</tr>
<tr>
<td>( A_g )</td>
<td>250000 mm²</td>
<td>387.50 in²</td>
</tr>
<tr>
<td>Dia hoop</td>
<td>10 mm</td>
<td>0.39 in</td>
</tr>
<tr>
<td>A hoop</td>
<td>78.54 mm²</td>
<td>0.12 in²</td>
</tr>
<tr>
<td>spacing</td>
<td>100 mm</td>
<td>3.94 in</td>
</tr>
<tr>
<td>( d )</td>
<td>445.5 mm</td>
<td>17.54 in</td>
</tr>
<tr>
<td>Dia Bar</td>
<td>22 mm</td>
<td>0.87 in</td>
</tr>
<tr>
<td>A bar</td>
<td>380.13 mm²</td>
<td>0.59 in²</td>
</tr>
</tbody>
</table>

Beam(s) Connected:

<table>
<thead>
<tr>
<th>Item</th>
<th>Value</th>
<th>Conversion</th>
</tr>
</thead>
<tbody>
<tr>
<td>bw</td>
<td>300 mm</td>
<td>11.81 in</td>
</tr>
<tr>
<td>h</td>
<td>600 mm</td>
<td>23.62 in</td>
</tr>
<tr>
<td>h clear</td>
<td>2400 mm</td>
<td>94.49 in</td>
</tr>
<tr>
<td>R</td>
<td>8</td>
<td></td>
</tr>
<tr>
<td>I</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td>( C_d )</td>
<td>5.5</td>
<td></td>
</tr>
</tbody>
</table>

Strength check:

Flexural strength

Column strength:

\( M_{n,\text{col}}^{\text{top}} = 346 \text{ kN-m} \)
\( M_{n,\text{col}}^{\text{bottom}} = 429 \text{ kN-m} \)
\( \mu = 200 \text{ kN-m} \)

\( \Phi M_n = 225 > \mu = 200 \)  \( \text{OK} \)
Beam(s) strength:

\[ M_{n,beam}^* = 386 \text{ kN-m} \]
\[ M_{n,beam} = 572 \text{ kN-m} \]

S21.6.3.1

Ast\(\geq 0.01Ag\)

\[ \text{# of bars} = 10 \]
\[ Ast = 3801.33 \text{ mm}^2 = 5.89 \text{ in}^2 \]

\[ Ast = 3801.33 > 0.01Ag = 2500.00 \text{ OK} \]

Axial Force ratio

\[ P_{\text{total}} = 772.02 \text{ kN} = 173.57 \text{ kips} \]
\[ P/f'cAg = 0.114 \]

Shear strength

1) \( Ve = 2\times M_{pr,\text{col}}/h \)

\[ M_{pr,\text{col}}^{\text{top}} = 432.51 \text{ kN-m} \]

2) \( Ve = (M_{pr,beam(+)}+M_{pr,beam(-)})/h \)

\[ M_{pr,\text{col}}^{\text{bottom}} = 536.75 \text{ kN-m} \]

\[ V_{e}^{(1)} = 403.86 \text{ kN} = 90.80 \text{ kips} \]
\[ V_{e}^{(2)} = 200.89 \text{ kN} = 45.16 \text{ kips} \]
\[ V_{e}^{(2)} = 298.02 \text{ kN} = 67.00 \text{ kips} \]
\[ V_{u} = 98.31 \text{ kN} = 22.10 \text{ kips} \]

If \( Ve/Vu > 0.5 \) & \( P < Agf'c/20 \) --> ignore \( Vc \)

\[ \text{current } Ve/Vu = 2.04 > \text{ limit } Ve/Vu = 0.5 \text{ OK} \]
\[ P = 772.02 > Agf'c/20 = 337.5 \text{ NOT OK} \]

Bottom section

\[ Vc = 192.18 \text{ kN} = 43.21 \text{ kips} \]
\[ \# \text{ of hoops} = 4 \]
\[ Vs = Ay*fy*d/s \]
\[ Vs = 482.85 \text{ kN} = 108.56 \text{ kips} \]
\[ Vn = Vc + Vs = 675.04 \text{ kN} = 151.76 \text{ kips} \]
\[ \Phi V_n = 506 > Ve = 298.02 \text{ OK} \]

If \( Vs < 4(bd)\sqrt{f'c} ; s < (d/2 ; 24") \)
If \( Vs < 4(bd)\sqrt{f'c} ; s < (d/4 ; 12") \)
**check if \( V_s < 4(bd)\sqrt{f'(c)} \)**

\[
V_s = 109 > 4\sqrt{f'(c)}bd = 86
\]

\[
s = \min\left(\frac{d}{2}; 24"\right) = 111.38 \text{ mm} \quad 4.38 \text{ in}
\]

\[
\text{current spacing} = 100 \text{ mm} \quad 3.94 \text{ in}
\]

\[
\text{current spacing} = 100 < s, \text{min} = 111 \quad \text{OK}
\]

**\( V_{s,\text{max}} = 8(bd)\sqrt{f'(c)} \)**

\[
V_s = 109 < 8\sqrt{f'(c)}bd = 173 \quad \text{OK}
\]

**Top section**

\[
V_c = 192.18 \text{ kN} \quad 43.21 \text{ kips}
\]

\[
\# \text{ of hoops} = 3
\]

\[
V_s = A_v f_y d/s \quad V_s = 362.14 \text{ kN} \quad 81.42 \text{ kips}
\]

\[
V_n = V_c + V_s \quad V_n = 554.32 \text{ kN} \quad 124.62 \text{ kips}
\]

\[
\Phi V_n = 416 > Ve = 298.02 \quad \text{OK}
\]

*if \( V_s < 4(bd)\sqrt{f'(c)} ; s < (d/2 ; 24") \)*

*if \( V_s < 4(bd)\sqrt{f'(c)} ; s < (d/4 ; 12") \)*

**check if \( V_s < 4(bd)\sqrt{f'(c)} \)**

\[
V_s = 81 < 4\sqrt{f'(c)}bd = 86 \quad \text{OK}
\]

\[
s = \min\left(\frac{d}{2}; 24"\right) = 222.75 \text{ mm} \quad 8.77 \text{ in}
\]

\[
\text{current spacing} = 100 \text{ mm} \quad 3.94 \text{ in}
\]

\[
\text{current spacing} = 100 < s, \text{min} = 223 \quad \text{OK}
\]

**\( V_{s,\text{max}} = 8(bd)\sqrt{f'(c)} \)**

\[
V_s = 81 < 8\sqrt{f'(c)}bd = 173 \quad \text{OK}
\]

**Detailing:**

*521.6.4*

\[
\# \text{ of hoops} = 4
\]

\[
A_{sh} = 314.16 \text{ mm}^2 \quad 0.49 \text{ in}^2
\]
\[ bc = 417 \text{ mm} \quad 16.42 \text{ in} \]
\[ Ach = 173889 \text{ mm}^2 \quad 269.53 \text{ in}^2 \]
\[ hx = 226 \text{ mm} \quad 8.90 \text{ in} \]
\[ so = 144.8 \text{ mm} \quad 5.70 \text{ in} \]

\[ S21.6.4.1 \]
\[ l_o \leq \min \left( \text{member depth; } \frac{1}{6} \times \text{clear height; } 18'' \right) \]
\[ l_o = 400.00 \text{ mm} \quad 15.75 \text{ in} \]

spacing same everywhere-->

current lo= 2400 mm \quad 94.49 in

\[ \text{current lo}= 2400 \]
\[ > \]
\[ \text{lo,min} = 400 \quad \boxed{\text{OK}} \]

Within \( l_o \):
\[ s \leq \min \left( \frac{h}{4}; 6db; \text{so; } 6'' \right) \]
\[ s = \min \left( \frac{h}{4}; 6db; \text{so; } 6'' \right) \]
\[ so; 6'' \]
\[ 125 \text{ mm} \quad 4.92 \text{ in} \]

current spacing= 100 mm \quad 3.94 in

\[ \text{current spacing}= 100 \]
\[ < \]
\[ s,\text{min} = 125 \quad \boxed{\text{OK}} \]

\[ S21.6.4.4 \]
\[ s \left(1\right) \leq \frac{A_{sh}}{0.3 \cdot b_c f'c/f_v \left( A_g/A_{ch} - 1 \right)} \]
\[ s \left(2\right) \leq \frac{A_{sh}}{0.09 \cdot b_c f'c/f_v} \]
\[ s^{(1)} = 73.31 \text{ mm} \quad 2.89 \text{ in} \]
\[ s^{(2)} = 106.96 \text{ mm} \quad 4.21 \text{ in} \]

current spacing= 100 \quad >

\[ s,\text{min} = 73 \quad \boxed{\text{NOT OK}} \]

Beyond \( l_o \):
\[ s = \min \left( 6db; 6'' \right) \]
\[ 132 \text{ mm} \quad 5.20 \text{ in} \]

current spacing= 100 mm \quad 3.94 in

\[ \text{current spacing}= 100 \]
\[ < \]
\[ s,\text{min} = 132 \quad \boxed{\text{OK}} \]
Drift check: (ASCE7-05 12.12)

Δs shall be <= 0.02/ρ = 0.015

<table>
<thead>
<tr>
<th>Floor</th>
<th>h (mm)</th>
<th>δxe (mm)</th>
<th>δx (mm)</th>
<th>Δi</th>
<th>Status</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>12000</td>
<td>22.30</td>
<td>122.65</td>
<td>0.0068</td>
<td>OK</td>
</tr>
<tr>
<td>3</td>
<td>9000</td>
<td>18.60</td>
<td>102.3</td>
<td>0.0108</td>
<td>OK</td>
</tr>
<tr>
<td>2</td>
<td>6000</td>
<td>12.70</td>
<td>69.85</td>
<td>0.0134</td>
<td>OK</td>
</tr>
<tr>
<td>1</td>
<td>3000</td>
<td>5.40</td>
<td>29.7</td>
<td>0.0099</td>
<td>OK</td>
</tr>
</tbody>
</table>

Δtotal= 0.0102  OK
# INTERIOR COLUMNS

## Materials

Concrete: \( f'_c = \) 27 MPa \( \approx 3.9 \) ksi  
Steel: \( f_y = \) 345 MPa \( \approx 50.0 \) ksi

## 1C2 (Frame Direction) -- Interior Column

### Cross-section

- \( h_i = \) 3000 mm \( \approx 118.11 \) in  
- Total height = 12000 mm \( \approx 472.44 \) in  
- \( H_c = \) 500 mm \( \approx 19.69 \) in  
- \( B_c = \) 500 mm \( \approx 19.69 \) in  
- \( A_g = \) 250000 mm\(^2\) \( \approx 387.50 \) in\(^2\)  
- Diahoop = 10 mm \( \approx 0.39 \) in  
- Ahoop = 78.54 mm\(^2\) \( \approx 0.12 \) in\(^2\)  
- spacing = 100 mm \( \approx 3.94 \) in  
- \( d = \) 445.5 mm \( \approx 17.54 \) in  
- DiaBar = 22 mm \( \approx 0.87 \) in  
- Abar = 380.13 mm\(^2\) \( \approx 0.59 \) in\(^2\)

### Beam(s) Connected:

- \( 2 \times 2G1 \)
  - \( bw = \) 300 mm \( \approx 11.81 \) in  
  - \( h = \) 600 mm \( \approx 23.62 \) in  
  - \( h_{clear} = \) 2400 mm \( \approx 94.49 \) in  
  - \( R = \) 8  
  - \( I = \) 1  
  - \( C_d = \) 5.5

### Strength check:

#### Flexural strength

Column strength: \( M_{n, col}^{top} = \) 486 kN-m  
\( M_{n, col}^{bottom} = \) 486 kN-m  
\( M_u = \) 205 kN-m

\[
\Phi M_n = 316 \quad > \quad M_u = 205 \quad \text{OK}
\]

Beam(s) strength: \( M_{n, beam}^* = \) 386 kN-m
M_{n,beam} = 572 \text{ kN-m}

S21.6.3.1
Ast >= 0.01Ag
# of bars = 10
Ast = 3801.33 \text{ mm}^2 \quad 5.89 \text{ in}^2

\text{OK}

Axial Force ratio
\begin{align*}
P_{\text{total}} &= 1222.22 \text{ kN} \quad 274.78 \text{ kips} \\
P/f'cAg &= 0.181
\end{align*}

Shear strength

1) \( V_e = 2*M_{pr, col}/h \) 
2) \( V_e = (M_{pr, beam(+) + M_{pr, beam(-)})/h \)

\begin{align*}
M_{pr, col}^{\text{top}} &= 607.38 \text{ kN-m} \\
M_{pr, col}^{\text{bottom}} &= 607.38 \text{ kN-m} \\
V_e^{(1)} &= 506.15 \text{ kN} \quad 113.79 \text{ kips} \\
V_e^{(2)} &= 498.91 \text{ kN} \quad 112.16 \text{ kips} \\
V_u &= 98.31 \text{ kN} \quad 22.10 \text{ kips}
\end{align*}

If \( V_e/V_u > 0.5 \) & \( P < Agf'c/20 \) --> ignore \( V_c \)

\begin{align*}
current \ V_e/V_u &= 5.07 \quad > \quad \text{Ve/Vu lim}= 0.5 \quad \text{OK} \\
P &= 1222.22 \quad > \quad Agf'c/20 = 337.5 \quad \text{NOT OK}
\end{align*}

\begin{align*}
V_c &= 192.18 \text{ kN} \quad 43.21 \text{ kips} \\
# \ of \ hoops &= 4 \\
V_c &= 192.18 \text{ kN} \quad 43.21 \text{ kips} \\
V_s &= 482.85 \text{ kN} \quad 108.56 \text{ kips} \\
V_n &= V_c + V_s = 675.04 \text{ kN} \quad 151.76 \text{ kips} \\
\Phi V_n &= 506 \quad > \quad Ve = 498.91 \quad \text{OK}
\end{align*}

if \( V_s < 4(bd)\sqrt{f'c} \); \( s < (d/2 \ ; 24") \)
if \( V_s < 4(bd)\sqrt{f'c} \); \( s < (d/4 \ ; 12") \)

check if \( V_s < 4(bd)\sqrt{f'c} \)
\begin{align*}
V_s &= 109 \quad > \quad 4\sqrt{f'c}bd = 86 \quad s < (d/4 \ ; 12") \quad \text{s < (d/4; 12")}
\end{align*}

s = \text{min}(d/2; 24") = 111.38 \text{ mm} \quad 4.38 \text{ in}
current spacing = 100 mm 3.94 in

current spacing = 100 < s, min = 111 OK

\[ V_{max} = 8(bd)\sqrt{f'c} \]
\[ V_s = 109 < 8\sqrt{f'c}bd 173 OK \]

**Detailing:**
*S21.6.4*

- # of hoops = 4
- \( Ash = 314.16 \text{ mm}^2 \quad 0.49 \text{ in}^2 \)
- \( bc = 417 \text{ mm} \quad 16.42 \text{ in} \)
- \( Ach = 173889 \text{ mm}^2 \quad 269.53 \text{ in}^2 \)
- \( hx = 240 \text{ mm} \quad 9.45 \text{ in} \)
- \( so = 140.1 \text{ mm} \quad 5.52 \text{ in} \)

*S21.6.4.1*

\( lo <= \min (\text{member depth; } 1/6*\text{clear height; } 18'') \)

\[ l_o = 400.00 \text{ mm} \quad 15.75 \text{ in} \]

spacing same everywhere -->

- current lo = 2400 mm 94.49 in
- current lo = 2400 > lo, min = 400 OK

**Within lo:**
*S21.6.4.3*

\( s = \min (h/4; 6db) \)

\( s <= \min (h/4; 6db; so; 6'') \)

\( \text{so; } 6'' \)

current spacing = 100 mm 3.94 in

- current spacing = 100 < s, min = 125 OK

*S21.6.4.4*

\( s (1) <= A_{sh} / (0.3 b_c f'_c f_y (A_g/A_{ch} - 1)) \)

\( s (2) <= A_{sh} / (0.09 b_c f'_c f_y) \)

\[ s^{(1)} = 73.31 \text{ mm} \quad 2.89 \text{ in} \]

\[ s^{(2)} = 106.96 \text{ mm} \quad 4.21 \text{ in} \]

current spacing = 100 > s, min = 73 NOT OK

**Beyond lo:**

\( s = \min (6db; 6'') \)

current spacing = 100 mm 3.94 in
current spacing = 100 < s,min = 132

2C2 (Frame Direction) -- Interior Column

Cross-section

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
<th>Conversion</th>
</tr>
</thead>
<tbody>
<tr>
<td>hi</td>
<td>3000 mm</td>
<td>118.11 in</td>
</tr>
<tr>
<td>Total height</td>
<td>12000 mm</td>
<td>472.44 in</td>
</tr>
<tr>
<td>Hc =</td>
<td>500 mm</td>
<td>19.69 in</td>
</tr>
<tr>
<td>Bc =</td>
<td>500 mm</td>
<td>19.69 in</td>
</tr>
<tr>
<td>Ag =</td>
<td>250000 mm²</td>
<td>387.50 in²</td>
</tr>
<tr>
<td>Diahoop =</td>
<td>10 mm</td>
<td>0.39 in</td>
</tr>
<tr>
<td>Ahoop =</td>
<td>78.54 mm²</td>
<td>0.12 in²</td>
</tr>
<tr>
<td>spacing =</td>
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<td>3.94 in</td>
</tr>
<tr>
<td>d =</td>
<td>445.5 mm</td>
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</tr>
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<td>DiaBar =</td>
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</tr>
<tr>
<td>Abar =</td>
<td>380.13 mm²</td>
<td>0.59 in²</td>
</tr>
</tbody>
</table>

Beam(s) Connected:

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
<th>Conversion</th>
</tr>
</thead>
<tbody>
<tr>
<td>bw =</td>
<td>300 mm</td>
<td>11.81 in</td>
</tr>
<tr>
<td>h =</td>
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</tr>
<tr>
<td>hclear =</td>
<td>2400 mm</td>
<td>94.49 in</td>
</tr>
<tr>
<td>R =</td>
<td>8</td>
<td></td>
</tr>
<tr>
<td>l =</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td>Cd =</td>
<td>5.5</td>
<td></td>
</tr>
</tbody>
</table>

Strength check:

Flexural strength

Column strength:

\[ M_{n,\text{col}}^{\text{top}} = 456 \text{ kN-m} \]
\[ M_{n,\text{col}}^{\text{bottom}} = 456 \text{ kN-m} \]
\[ M_u = 187 \text{ kN-m} \]

\[ \phi M_n = 296 \]

Beam(s) strength:

\[ M_{n,\text{beam}}^+ = 380 \text{ kN-m} \]
\[ M_{n,\text{beam}}^- = 527 \text{ kN-m} \]

S21.6.3.1

Ast >= 0.01Ag

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
<th>Conversion</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ast =</td>
<td>3801.33 mm²</td>
<td>5.89 in²</td>
</tr>
</tbody>
</table>

# of bars = 10
Axial Force ratio

\[ P_{\text{total}} = 919.65 \text{ kN} \quad 206.76 \text{ kips} \]
\[ P/f'cA_g = 0.136 \]

Shear strength

1) \( V_e = 2 \times \text{M}_{\text{pr, col}}/h \)
\[ \text{M}_{\text{pr, col}} = 569.80 \text{ kN-m} \]
2) \( V_e = (\text{M}_{\text{pr, beam(+)}} + \text{M}_{\text{pr, beam(-)}})/h \)
\[ \text{M}_{\text{pr, col\ bottom}} = 569.80 \text{ kN-m} \]
\[ V_e^{(1)} = 474.84 \text{ kN} \quad 106.75 \text{ kips} \]
\[ V_e^{(2)} = 472.50 \text{ kN} \quad 106.23 \text{ kips} \]
\[ V_u = 89.01 \text{ kN} \quad 20.01 \text{ kips} \]

If \( Ve/Vu > 0.5 \) \& \( P < Agf'c/20 \) --> ignore \( V_c \)

current \( Ve/Vu = 5.31 \) > limit \( Ve/Vu = 0.5 \) \[ OK \]
\[ P = 919.65 \text{ kN} \quad > \quad Agf'c/20 = 337.5 \] \[ NOT OK \]

\[ V_c = 192.18 \text{ kN} \quad 43.21 \text{ kips} \]
\[ # \text{ of hoops} = 4 \]
\[ V_s = Av*fy*d/s \]
\[ Vs = 482.85 \text{ kN} \quad 108.56 \text{ kips} \]
\[ V_n = V_c + V_s = 675.04 \text{ kN} \quad 151.76 \text{ kips} \]
\[ \Phi V_n = 506 \quad > \quad Ve = 472.50 \] \[ OK \]

if \( Vs < 4(bd)sqrt(f'c) \) ; \( s < (d/2 ; 24") \)
if \( Vs < 4(bd)sqrt(f'c) \) ; \( s < (d/4 ; 12") \)

check if \( Vs < 4(bd)sqrt(f'c) \)

\[ Vs = 109 \quad > \quad 4sqrt(f'c)bd = 86 \quad s < (d/4 ; 12) \]
\[ s = \min(d/2 ; 24") = 111.38 \text{ mm} \quad 4.38 \text{ in} \]
\[ \text{current spacing} = 100 \text{ mm} \quad 3.94 \text{ in} \]
\[ \text{current spacing} = 100 < s, \min = 111 \] \[ OK \]

\( V_{\text{max}} = 8(bd)sqrt(f'c) \)
Vs = 109 < 8sqrt(f'c)bd 173  OK

**Detailing:**

S21.6.4

# of hoops = 4

Ash = 314.16 mm² 0.49 in²
bc = 407 mm 16.02 in
Ach = 165649 mm² 256.76 in²

hx = 163 mm 6.42 in
so = 165.8 mm 6.53 in

S21.6.4.1

lo <= min (member depth; 1/6*clear height; 18")

lo = 400.00 mm 15.75 in

spacing same everywhere-->
current lo= 2400 mm 94.49 in

current lo= 2400 > lo,min = 400  OK

**Within lo:**

s <= min (h/4; 6db; so; 6")

s = min (h/4; 6db; so; 6")
current spacing = 125 mm 4.92 in

current spacing = 100 mm 3.94 in

current spacing = 100 < s,min = 125  OK

S21.6.4.4

s (1) <= Ash / (0.3 b_c f'c/f_y (A_y/A_sha - 1))

s (2) <= Ash / (0.09 b_c f'c/f_y)

s(1) = 64.56 mm 2.54 in
s(2) = 109.59 mm 4.31 in

current spacing = 100 > s,min = 65  NOT OK

**Beyond lo:**

s = min (6db; 6")
current spacing = 100 mm 3.94 in
current spacing = 100 < s,min = 132 OK

3C2 (Frame Direction) -- Interior Column

Cross-section

hi = 3000 mm 118.11 in
Total height = 12000 mm 472.44 in
Hc = 500 mm 19.69 in
Bc = 500 mm 19.69 in
Ag = 250000 mm² 387.50 in²
Dia hoop = 10 mm 0.39 in
A hoop = 78.54 mm² 0.12 in²
spacing = 100 mm 3.94 in
d = 445.5 mm 17.54 in
Dia bar = 22 mm 0.87 in
A bar = 380.13 mm² 0.59 in²

Beam(s) Connected:
2 x 4G1
bw = 300 mm 11.81 in
h = 600 mm 23.62 in
h clear = 2400 mm 94.49 in
R = 8
l = 1
C d = 5.5

Strength check:

Flexural strength

Column strength: 
\[ M_{n,\text{col}}^{\text{top}} = 442 \text{ kN-m} \]
\[ M_{n,\text{col}}^{\text{bottom}} = 442 \text{ kN-m} \]
\[ M_u = 153 \text{ kN-m} \]
\[ \Phi M_n = 287 > Mu = 153 \text{ OK} \]

Beam(s) strength:
\[ M_{n,\text{beam}}^+ = 373 \text{ kN-m} \]
\[ M_{n,\text{beam}}^- = 475 \text{ kN-m} \]
S21.6.3.1

Ast>=0.01Ag

# of bars= 10

Ast = 3801.33 mm² 5.89 in²

Ast = 3801.33 > 0.01Ag = 2500.000 OK

Axial Force ratio

Ptotal= 620.64 kN 139.53 kips

P/f‘cAg= 0.092

Shear strength

1) Ve = 2*Mpr,col/h

Mpr,col top = 552.85 kN-m

Mpr,col bottom = 552.85 kN-m

Vₑ(1) = 460.71 kN 103.58 kips

Vₑ(2) = 441.77 kN 99.32 kips

Vu = 70.07 kN 15.75 kips

If Ve/Vu > 0.5 & P<Agf’c/20 --> ignore Vc

current Ve/Vu = 6.30 > limit Ve/Vu 0.5 OK

P = 620.64 > Agf’c/20 = 337.5 NOT OK

Vc = 192.18 kN 43.21 kips

# of hoops= 2

Vs = Av*fy*d/s

Vs = 241.43 kN 54.28 kips

Vn= Vc+Vs

433.61 kN 97.48 kips

ΦVn = 325 < Ve = 441.77 NOT OK

if Vs< 4(bd)sqrt(f’c) ; s < (d/2 ; 24")

if Vs< 4(bd)sqrt(f’c) ; s < (d/4 ; 12")

check if Vs< 4(bd)sqrt(f’c)

Vs = 54 < 4sqrt(f’c)bd 86 s < (d/2 ; 24)

s = min(d/2; 24") = 222.75 mm 8.77 in
current spacing = 100 mm 3.94 in

\[ \text{current spacing} = 100 \quad < \quad s, \text{min} = 223 \quad \text{OK} \]

\[ V_{\text{max}} = 8(bd)\sqrt{f'c} \]

\[ V_s = 54 \quad < \quad 8\sqrt{f'c}bd = 173 \quad \text{OK} \]

**Detailing:**

**S21.6.4**

# of hoops = 2

\[ \text{Ash} = 157.08 \quad \text{mm}^2 \quad 0.24 \quad \text{in}^2 \]
\[ \text{bc} = 409 \quad \text{mm} \quad 16.10 \quad \text{in} \]
\[ \text{Ach} = 167281 \quad \text{mm}^2 \quad 259.29 \quad \text{in}^2 \]

\[ \text{hx} = 210 \quad \text{mm} \quad 8.27 \quad \text{in} \]
\[ \text{so} = 150.1 \quad \text{mm} \quad 5.91 \quad \text{in} \]

**S21.6.4.1**

\[ l_o = 400.00 \quad \text{mm} \quad 15.75 \quad \text{in} \]

spacing same everywhere-->

\[ \text{current } l_o = 2400 \quad \text{mm} \quad 94.49 \quad \text{in} \]

\[ \text{current } l_o = 2400 \quad > \quad l_o, \text{min} = 400 \quad \text{OK} \]

**Within lo:**

\[ s \leq \min (h/4; 6\text{db}; so; 6") \]

\[ s \leq \min (h/4; 6\text{db}; so; 6") = 125 \quad \text{mm} \quad 4.92 \quad \text{in} \]

\[ \text{current spacing} = 100 \quad \text{mm} \quad 3.94 \quad \text{in} \]

\[ \text{current spacing} = 100 \quad < \quad s, \text{min} = 125 \quad \text{OK} \]

**S21.6.4.4**

\[ s(1) \leq \frac{A_{\text{sh}}}{0.3 \: b_c \: f'c / f_y} (A_{\text{sh}} / A_{\text{ch}} - 1) \]

\[ s(2) \leq \frac{A_{\text{sh}}}{0.09 \: b_c \: f'c / f_y} \]

\[ s^{(1)} = 33.08 \quad \text{mm} \quad 1.30 \quad \text{in} \]
\[ s^{(2)} = 54.53 \quad \text{mm} \quad 2.15 \quad \text{in} \]

\[ \text{current spacing} = 100 \quad > \quad s, \text{min} = 33 \quad \text{NOT OK} \]
Beyond \( lo: \)

\[
\begin{align*}
s &= \text{\( min \)} (6db; 6") \\
&= 132 \text{ mm} \\
&= 5.20 \text{ in}
\end{align*}
\]

\[
\begin{align*}
\text{current spacing} &= 100 \text{ mm} \\
&= 3.94 \text{ in}
\end{align*}
\]

\[
\begin{align*}
\text{current spacing} &= 100 \text{ mm} < \text{s,\( \text{min} = 132 \text{ mm} \)} \\
&= \text{OK}
\end{align*}
\]

**Beam Column Joint - G1-C2-G1 - frame direction (case 1)**

**Materials**

Concrete: \( f'_c = 3.9 \text{ ksi} \)

Steel: \( f_y = 50 \text{ ksi} \)

**Cross-section**

\[
\begin{align*}
B_{\text{slab}} &= 66 \text{ in} \\
B &= 11.81 \text{ in} \\
d &= 22.10 \text{ in}
\end{align*}
\]

**Nominal Moment Capacity of Beams - G1**

\[
\begin{align*}
&\#7 \\
M_{n,1}^+ &= A_{s,1} = 0.60 \text{ in}^2 \\
&= A_s = 1.8 \text{ in}^2 \\
a &= A_s f_y / (0.85 f'_c B) = 0.41 \text{ in} \\
M_{n,1}^+ &= 1970.49 \text{ in-kip} \\
M_{n,1}^+ &= 164.21 \text{ ft-kip}
\end{align*}
\]

\[
\begin{align*}
&\#7 \\
M_{n,2}^- &= A_{s,2} = 0.6 \text{ in}^2 \\
&= A_s = 2.4 \text{ in}^2 \\
a &= A_s f_y / (0.85 f'_c B) = 3.07 \text{ in} \\
M_{n,2}^- &= 2468.09 \text{ in-kip} \\
M_{n,2}^- &= 205.67 \text{ ft-kip}
\end{align*}
\]

\[
\begin{align*}
M_{n,pr}^+ &= 205.26 \text{ ft-kip} \\
M_{n,pr}^- &= 257.09 \text{ ft-kip}
\end{align*}
\]
Interior Connection G1-C2-G1

- \( h_{\text{column}} = 19.68 \text{ in} \)
- \( b_{\text{col}} = 19.68 \text{ in} \)
- \( b_w = 11.81 \text{ in} \)
- \( x = 3.94 \text{ in} \)
- \( b_{\text{eff}} = 19.68 \text{ in} \)

**#7**
Long beam bars: 19 bars

- \( d_b = 0.875 \text{ in} \)
- \( A_{sb,1} = 1.8 \text{ in}^2 \)
- \( A_{sb,2} = 2.4 \text{ in}^2 \)
- \( f'c = 3900 \text{ psi} \)
- \( \gamma_V = 12 \)

(beam frame into three faces of a column but the beam width is less than \(3/4\) of the column width)

- \( M_{pr,b1} = 257.09 \text{ ft-kip} \)
- \( M_{pr,b2} = 205.26 \text{ ft-kip} \)
- \( h_{\text{clear}} = 8.86 \text{ in} \)

\[ M_{C1} = M_{C2} = M_C = \frac{(M_{pr,b1} + M_{pr,b2})}{2} = 231.18 \text{ ft-kip} \]

\[ V_{C1} = \frac{M_{C1}}{(h_{\text{clear}}/2)} = 52.19 \text{ kip} \]

**Joint Shear Demand**

\[ V_{u,\text{joint}} = 1.25 f_y A_{sb,1} + 1.25 f_y A_{sb,2} - V_{C1} = 210.31 \text{ kip} \]

- \( A_y = 387.30 \text{ in}^2 \)
- \( \theta_V = 0.85 \)
- \( \theta_V \gamma V_n = \theta_V (f'_c)^{0.5} A_y = 246.71 \text{ ksi} \)

\[ V_{u,\text{joint}} < \theta_V \gamma V_n \Rightarrow \text{OK} \]

**Joint Detailing Requirements**
\[
b_w = 11.81 < 3/4 b_{col} = 14.76
\]

\[\Rightarrow \text{ Required transferse reinforcement = 100\% Ash}\]

**Column - C2**

<table>
<thead>
<tr>
<th>Bars</th>
<th>(A_{sh})</th>
<th>(A_{ch})</th>
<th>(A_g)</th>
<th>(b_c)</th>
<th>(h_x)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3 #3 bars</td>
<td>0.33 \text{in}^2</td>
<td>216.97 \text{in}^2</td>
<td>387.30 \text{in}^2</td>
<td>14.56 \text{in}</td>
<td>7.905 \text{in}</td>
</tr>
</tbody>
</table>

**longitudinal column bars:** #7

<table>
<thead>
<tr>
<th>(d_{b,\text{col}})</th>
<th>(s_o)</th>
<th>(s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.875 \text{in}</td>
<td>6.03 \text{in}</td>
<td>(s &lt; A_{sh} / (0.3 \cdot b_c \cdot f'<em>c / f_y \cdot (A_g / A</em>{ch} - 1)) = 1.23 \text{in})</td>
</tr>
</tbody>
</table>

\[s = \min(b/4; 6d_b; s_o; 6") = 4.92 \text{in}\]

**Actual spacing in the structure:**

| \(s_A\) | 5.52 \text{in} |

**Joint Anchorage Requirements**

\[M_{n^+} / M_{n^-} = 0.80 \text{ > 0.5, OK}\]

Beam longitudinal reinforcement should be extended to the far face of the confined column and anchored in tension.

\[l_{dh} = f_y \cdot d_b / (65 \cdot (f'_c)^{0.5}) = 10.78 \text{ > 8 db = 7.0 or 6"}\]

\[l_{dh,\text{req}} = 6 " \text{, } l_{dh,\text{act}} = 14 " \text{ OK}\]

**Beam Column Joint - G1 - C1 - frame direction (case 2)**

**Materials**

Concrete: \(f'_c = 3.9 \text{ksi}\)
Steel:

\[ f_y = 50 \text{ ksi} \]

**Cross-section**

\[ B_{slab} = 66 \text{ in} \]
\[ B = 11.81 \text{ in} \]
\[ d = 22.10 \text{ in} \]

**Nominal Moment Capacity of Beams - G1**

\[ M_{n,+} = 1970.49 \text{ in-kip} \]
\[ M_{n,-} = 164.21 \text{ ft-kip} \]

\[ \#7 \]
\[ n = 3 \]
\[ A_{s,1} = 0.60 \text{ in}^2 \]
\[ A_s = 1.80 \text{ in}^2 \]
\[ a = \frac{A_s f_y}{(0.85 f'_c B)} = 0.41 \text{ in} \]

\[ M_{n,+} = 2468.09 \text{ in-kip} \]
\[ M_{n,-} = 205.67 \text{ ft-kip} \]

\[ \#7 \]
\[ n = 4 \]
\[ A_{s,1} = 0.60 \text{ in}^2 \]
\[ A_s = 2.4 \text{ in}^2 \]
\[ a = \frac{A_s f_y}{(0.85 f'_c B)} = 3.07 \text{ in} \]

**Exterior Connection G1 - C1**

\[ h_{column} = 19.68 \text{ in} \]
\[ b_{col} = 19.68 \text{ in} \]
\[ b_w = 11.81 \text{ in} \]
\[ x = 3.94 \text{ in} \]
\[ b_{eff} = 19.68 \text{ in} \]

\[ \#7 \]
\[ d_b = 0.875 \text{ in} \]
\[ A_{sb,2} = 2.40 \text{ in}^2 \]
\[ f'_c = 3900 \text{ psi} \]
\[ \gamma_v = 12 \]
(beams frame into two faces of a column)

\[ M_{pr,b1} = 257.09 \text{ ft-kip} \]
\[ h_{\text{clear}} = 8.86 \text{ in} \]
\[ M_{C1} = M_{C2} = M_C = \frac{(M_{pr,b})}{2} = 128.55 \text{ ft-kip} \]
\[ V_{C1} = M_{C1} / (h_{\text{clear}}/2) = 29.02 \text{ kip} \]

**Joint Shear Demand**

\[ V_{u,\text{joint}} = 1.25 f_y A_{sb,2} - V_{C1} = 120.98 \text{ kip} \]
\[ A_f = 387.30 \text{ in}^2 \]
\[ \theta_V = 0.85 \]
\[ \theta_V V_n = \theta_V \gamma_V (f_c')^{0.5} A_f = 246.71 \text{ ksi} \]

\[ V_{u,\text{joint}} < \theta_V V_n \Rightarrow \text{OK} \]

**Joint Detailing Requirements**

\[ b_w = 11.81 \text{ in} < \frac{3}{4} b_{\text{col}} = 14.76 \text{ in} \]

=> **Required transferse reinforcement = 100% Ash**

**Column - C1**

3 #3 bars

\[ A_{sh} = 0.33 \text{ in}^2 \]
\[ A_{ch} = 216.97 \text{ in}^2 \]
\[ A_g = 387.30 \text{ in}^2 \]
\[ b_c = 14.56 \text{ in} \]
\[ h_x = 7.905 \text{ in} \]

longitudinal column bars: #7

\[ d_{b,\text{col}} = 0.875 \text{ in} \]
\[ s_o = 6.03 \text{ in} \]
\[ s < A_{sh} / (0.3 b_c f'_c / f_y (A_g/A_{ch} - 1)) = 1.23 \text{ in} \]
\[ s = \min(b/4; 6d_b; s_o; 6") = 4.92 \text{ in} \]

Actual spacing in the structure:

\[ s_A = 5.52 \text{ in} \]
Joint Anchorage Requirements

- beam longitudinal reinforcement should be extended to the far face of the confined column and anchored in tension.

\[ l_{dh} = \frac{f_y d_b}{(65 (f'_c)^{0.5})} = 10.78 \quad > 8 \quad \text{db} = 7.0 \quad \text{or 6”} \]

\[ l_{dh,req} = 10.78 \quad " \quad l_{dh,act} = 14 \quad " \]

OK
WALLS

Materials

Concrete: \( f'c = 27 \text{ MPa} \) \( 3.9 \text{ ksi} \)
Steel: \( f_y = 345 \text{ MPa} \) \( 50.0 \text{ ksi} \)

Cross-section

\[
egin{align*}
hi &= 3000 \text{ mm} \quad 118.11 \text{ in} \\
hw &= 12000 \text{ mm} \quad 472.44 \text{ in} \\
Lw &= 2500 \text{ mm} \quad 98.43 \text{ in} \\
tw &= 250 \text{ mm} \quad 9.84 \text{ in} \\
A_{cv} &= 625000 \text{ mm}^2 \quad 968.75 \text{ in}^2 \\
Diahoop &= 10 \text{ mm} \quad 0.39 \text{ in} \\
A_{hoop} &= 78.54 \text{ mm}^2 \quad 0.12 \text{ in}^2 \\
\text{hoop spacing} &= 100 \text{ mm} \quad 3.94 \text{ in} \\
\text{web transverse spacing (AXIS A)} &= 125 \text{ mm} \quad 4.92 \text{ in} \\
\text{web transverse spacing (AXIS C)} &= 200 \text{ mm} \quad 7.87 \text{ in} \\
\text{boundary width} &= 400 \text{ mm} \quad 15.75 \text{ in} \\
# \text{ of bars in the boundary} &= 6 \\
Diabar &= 19 \text{ mm} \quad 0.75 \text{ in} \\
A_{bar} &= 283.53 \text{ mm}^2 \quad 0.44 \text{ in}^2 \\
R &= 6 \\
l &= 1 \\
Cd &= 5
\end{align*}
\]

Strength check:

Flexural strength

\[
egin{align*}
M_n &= 2884 \text{ kN-m} \\
Mu &= 3569 \text{ kN-m} \\
\Phi M_n &= 2595.4 < Mu = 3569 \quad \text{NOT OK}
\end{align*}
\]

Shear strength

AXIS A

\[
\begin{align*}
\alpha_c &= 2 \\
# \text{ of hoops} &= 2 \\
\rho_t &= 0.0050 \\
\rho_t &= 0.0050 > \rho_{min} = 0.0025 \quad \text{OK}
\end{align*}
\]

\[
egin{align*}
V_n &= 1622.74 \text{ kN} \\
Vu &= 393 \text{ kN-m}
\end{align*}
\]
\( \Phi V_n = 1217 \quad > \quad V_u = 393 \quad \text{OK} \)

**AXIS C**

\[ \begin{align*}
\alpha c &= 2 \\
# \text{of hoops} &= 2 \\
\rho t &= 0.0031 \\
\rho t &= 0.0031 \quad > \quad \rho_{\text{min}} &= 0.0025 \quad \text{OK}
\end{align*} \]

\( V_n = 1216.42 \text{ kN} \quad 273.48 \text{ kips} \)
\( V_u = 393 \text{ kN-m} \)

\( \Phi V_n = 912 \quad > \quad V_u = 393 \quad \text{OK} \)

**Axial Force ratio**

\( P_{\text{total}} = 284.86 \text{ kN} \quad 64 \text{ kips} \)
\( P/f'cA_g = 0.017 \)

**Detailing:**

**Need for special boundary elements:**

**At design-based earthquake DBE**

(elastic displacement)

\( \delta_{xe} = 28.46 \text{ mm} \quad 1.12 \text{ in} \)
\( \delta_u = 142.32 \text{ mm} \quad 5.60 \text{ in} \)

\( \delta_u/\text{hw} \) shall not be less than 0.007-->

\[ \delta_u/\text{hw} = 0.0119 \]

check if \( c = \text{l}\text{w}/600(\delta_u/\text{hw}) \)

\( \text{l}\text{w}/600(\delta_u/\text{hw}) = 351.33 \text{ mm} \quad 13.83 \text{ in} \)

(from BIAx)

\( c = 243.50 \text{ mm} \quad 9.59 \text{ in} \)

\( c = 244 \quad < \quad c_{\text{lim}} = 351 \quad \text{BE NOT NEEDED} \)

**At maximum considered earthquake MCE**

(elastic displacement)

\( \delta_{xe} = 42.69 \text{ mm} \quad 1.68 \text{ in} \)
\( \delta_u = 213.47 \text{ mm} \quad 8.40 \text{ in} \)

\( \delta_u/\text{hw} \) shall not be less than 0.007-->

\[ \delta_u/\text{hw} = 0.0178 \]

check if \( c = \text{l}\text{w}/600(\delta_u/\text{hw}) \)

\( \text{l}\text{w}/600(\delta_u/\text{hw}) = 234.22 \text{ mm} \quad 9.22 \text{ in} \)
\( c = 243.50 \text{ mm} \quad 9.59 \text{ in} \)

\( c = 244 \quad > \quad \text{l}\text{w}/600(\delta_u/\text{hw}) = 234 \quad \text{BE NEEDED} \)
if boundary elements are needed, length of BE:

c' = larger of \{c - 0.1w, c/2\}

\[
c' = 121.75 \text{ mm}
\]

current BE length = 400 mm

if not needed, satisfy 21.9.6.5

21.9.6.5(a): if \( p > 400/f_y \); satisfy 21.6.4.2 and 21.9.6.4(a); \( s < 8" \)

\[
\frac{400}{f_y} = 0.0080
\]

\[
p = 0.0170 > \frac{400}{f_y} = 0.0080
\]

\rightarrow 21.6.4.2 and 21.9.6.4(a); \( s < 8" \)

21.9.6.4(a): c' = larger of \{c - 0.1w, c/2\}

\[
c' = 121.75 \text{ mm}
\]

current BE length = 400 mm

21.6.4.2: \( h_x < 14" \)

1st floor

\[
h_x = 183.0 \text{ mm}
\]

\[
7.20 \text{ in}
\]

\[
current \ h_x = 183.0 < \ h_{x\text{limit}} = 355.6 \ \text{OK}
\]

upper floors

\[
h_x = 275.0 \text{ mm}
\]

\[
10.83 \text{ in}
\]

\[
current \ h_x = 275.0 < \ h_{x\text{limit}} = 355.6 \ \text{OK}
\]

check spacing: \( s < 8\ \text{db}; 8" \)

\[
s = 100 \text{ mm}
\]

\[
current \ s = 100 < 8\text{ in}= 203.2 \ \text{OK}
\]

\[
current \ s = 100 < 8\text{db}= 152 \ \text{OK}
\]

Hoop reinforcement: \( A_{sh} \):

in \( x\)-dir:

\[
# \text{ of hoops} = 2
\]

\[
bc = 163 \text{ mm} \quad 6.42 \text{ in}
\]

\[
A_{ch} = 51345 \text{ mm}^2 \quad 79.58 \text{ in}^2
\]

\[
A_{g} = 100000 \text{ mm}^2 \quad 155.00 \text{ in}^2
\]
current $A_{sh} = 157 \text{ mm}^2 = 0.24 \text{ in}^2$

(eq.21-4) $A_{sh} \geq 0.3 s b_c f'c / f_y (A_g / A_{ch} - 1)$

\[
\begin{align*}
\text{min } A_{sh} &= 363 \text{ mm}^2 = 0.56 \text{ in}^2 \\
\text{current } A_{sh} &= 157 \quad < \quad \text{min } A_{sh} = 362.65 \quad \text{NOT OK}
\end{align*}
\]

(eq. 21-5) $A_{sh} \geq 0.09 s b_c f'c / f_y$

\[
\begin{align*}
\text{min } A_{sh} &= 115 \text{ mm}^2 = 0.18 \text{ in}^2 \\
\text{current } A_{sh} &= 157 \quad > \quad \text{min } A_{sh} = 114.81 \quad \text{OK}
\end{align*}
\]

\textit{in y-dir:}

\# of hoops= 3
$bc= 315 \text{ mm} = 12.40 \text{ in}$
\text{current } A_{sh} = 236 \text{ mm}^2 = 0.37 \text{ in}^2

(eq.21-4) $A_{sh} \geq 0.3 s b_c f'c / f_y (A_g / A_{ch} - 1)$

\[
\begin{align*}
\text{current } A_{sh} &= 236 \quad < \quad \text{min } A_{sh} = 700.82 \quad \text{NOT OK}
\end{align*}
\]

(eq. 21-5) $A_{sh} \geq 0.09 s b_c f'c / f_y$

\[
\begin{align*}
\text{current } A_{sh} &= 236 \quad > \quad \text{min } A_{sh} = 114.81 \quad \text{OK}
\end{align*}
\]

\textit{Drift check: (ASCE7-05 12.12)}

$\Delta s$ shall be $\leq 0.02/\rho = 0.015$

<table>
<thead>
<tr>
<th>Floor</th>
<th>$h$ (mm)</th>
<th>$\delta_{xe}$ (mm)</th>
<th>$\delta x$ (mm)</th>
<th>$\Delta i$</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>12000</td>
<td>28.46</td>
<td>142.32</td>
<td>0.0166</td>
</tr>
<tr>
<td>3</td>
<td>9000</td>
<td>18.49</td>
<td>92.45</td>
<td>0.0151</td>
</tr>
<tr>
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$\Delta_{total}= 0.0119 \quad \text{OK}$

NOTE: Member capacities are calculated based on SD345 strength for all reinforcement.
Appendix C

C.1 CONSTRUCTION PROCESS

Figure C.1 Construction of RC specimen versus PT specimen.
Figure C.2 Construction of RC specimen.
Figure C.3  Construction of PT specimen (column).
Figure C.4  Construction of PT specimen (beam and slab).
Figure C.5  Construction of PT specimen (walls).
Figure C.6 Construction of PT specimen (walls).
Appendix D

D.1 INSTRUMENTATION

![Diagram of instrument locations](image)

(i) Plan

(ii) Elevation

(a) RC specimen

(b) PC specimen

Displacement transducer

Accelerometer

Figure D.1 Measurements.
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</table>
層間変位計測用架台U：2台
(下は既存架台使用)
層間変位計測用架台B：1台

1F撤去図（見下げ）（PT）
三层変位計測用架台U：2台
（下は既存架台使用）

四層変位計測用架台U：2台
四層変位計測用架台B：1台

3 F梁伏樑（見下ろしPT）
RF梁床図  (見下げ) (PT)
層間変位計測用架台U : 2台
(下は既存架台使用)
層間変位計測用架台B : 1台
※機械基礎は3階のみ

層間変位計測用架台U : 2台
(下は既存架台使用)

層間変位計測用架台B : 1台
層間変位計測用架台 U：2台
（下は既存架台使用）
層間変位計測用架台 B：1台

※機械基礎は3階のみ

平面図(RC) (a=1/100)
1 概要

独立行政法人防災科学技術研究所殿がE-ディフェンスにて実施する「高性能RC建物実験において、実験データ収集のための計測準備作業（計測機器設置撤去・カメラ設置撤去）についての作業を行った。

本資料は、加振実験前に実施する詳細計画、センサ等の取付け、ケーブル配線、ラインチェック、ノイズ確認、信号確認、カメラ設置確認・実験終了後の撤収作業を実施した報告書である。

2 実施場所

〒673-0515 兵庫県三木市志染町三津田西亀居1501-21

独立行政法人 防災科学技術研究所 兵庫耐震工学研究センター(E-ディフェンス)

3 実施工程

実施工程表をページ3に示し、加振実験日を下表に示す。

<table>
<thead>
<tr>
<th>試験日</th>
<th>加振内容</th>
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<tbody>
<tr>
<td>1日目</td>
<td>12月13日 JMA神戸波10%・25%・50%</td>
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<td>2日目</td>
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<td>12月17日 JR長崎波40%・80% プレス公開試験</td>
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試験体設置状況
実施体制及び安全留意事項
実施体制及び緊急連絡先をページ41に、作業上の安全留意事項をページ9に示す。

実施内容
(1) 計測に必要なセンサ機器類を倉庫から搬出し、負数確認やケーブル伸ばし等の事前準備を行った。
(2) 計測センサ及び小型治具類を指定された箇所に取付け、ケーブルをジャックショックス
（JB）まで配線接続した。（カメラ類も同様である）
(3) 計測センサの取付け状況や、配線の状態が適正であるかを判断するために、信号の確認を行った。
(4) カメラの設置を行い、映像集録までの確認を行った。
(5) 実験終了後、計測センサや機器類を取外し、整理後作業日定の箇所に返納した。

詳細資料
(1) 震動台の方向と位置関係及びジャックショックス(JB)の配置図をページ3に示す。
(2) ジャックショックス(JB)の配置図をページ11に示す。
(3) 計測点数一覧表をページ8に示す。
(4) 計測センサー一覧及びカメラ一覧をページ9～10に示す。
(5) 計測ブロック線図をページ11～16に示す。
(6) 計測センサマニュアルリスト一覧をページ17～18に示す。
(7) 計測センサマニュアルリスト（各JB）をページ19～31に示す。
(8) センサの設置位置図をページ32～41に示す。
(9) センサの寸法計測図をページ42～59に示す。
(10) カメラの設置位置図をページ56～58に示す。
(11) プリッジボックス接線図をページ59～61に示す。
(12) プリッジボックス設置位置図をページ69に示す。
(13) ケーブル準備表をページ70～73に示す。
(14) ケーブル配線ルート図をページ74～88に示す。
(15) 試験体定点写真を添付1に示す。
(16) センサー写真・作業風景写真・試験体写真を添付2に示す。
(17) 屋根変位計測架台の製作・設置・撤去及び使用材料試験の実施結果を添付3に示す。((後部...
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<td>3) センサー取付</td>
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<td>4) リード線の解線</td>
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<td>5) リード線の調整</td>
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<tr>
<td>6) 周波数、圧力、温度測定</td>
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<tr>
<td>7) 電流の対応</td>
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</tr>
<tr>
<td>8) センサー、リード線の取付</td>
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<td>9) 整理、送付</td>
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<td>10) 型式書付成</td>
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実施工程表
実施体制表

（独）防災科学技術研究所

震動実験総合エンジニアリング（株）

国際振動計器（株）

（株）ニューテック 原田工業（株）

緊急連絡先

| （独）防災科学技術研究所 E-Defense | 0794-85-7654 |
| 震動実験総合エンジニアリング（株） | 0794-87-8365 |
| 国際振動計器（株） | 079-443-2617 |
| （株）ニューテック | 079-436-6200 |
| 原田工業（株） | 079-442-4297 |
| 三木警察署 | 0794-82-0110 |
| 三木消防署 | 0794-82-0119 |
| 加古川労働基準監督 | 079-422-5001 |
安全留意事項

1) 作業開始時には安全教育を必ず受講し、決められた事項を遵守すること。 
2) 毎朝朝礼に参加し、TBMを行うこと。 
3) 火炎を発生させる場合は火気使用配慮を提出すること。（消火器等用意する） 
4) 電源ドラム、電動工具等電気機器類は点検を受けた物を使用する。 
5) 決められたコンセントボックスを使用する。 
6) 1.5m以上の高所作業では必ず安全帯を使用する。 
7) 安全帯のフックは腰より高い位置に掛ける。 
8) 玉掛け作業時は吊荷の下に入らないこと。 
9) 上下作業は同時並行的に行わない。 
10) 一人作業は行わない。 
11) 視認所外の場所では視認を行わない。 
12) ハンダガチ等、火気使用時は、火気使用後30分以上経過後に残火の再確認をする。 
13) 手元足元の状況を確認し、作業を行う。（周囲の確認） 
14) 照明を確保し、明るい場所で作業を行うこと。 
15) 作業場所の整理整頓清掃は進んで行うこと。 
16) 決められた交通ルールを遵守すること。
電動台の方向と位置関係

ジャンクションボックス（JB）の様子
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<td>300</td>
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<td>2,000</td>
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<tr>
<td>C</td>
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<td>D</td>
<td>70kg</td>
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<td>3,700</td>
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<td>E</td>
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</table>

合計：
- 重量：300kg
- 購入価格：11,000円
- 売出し価格：13,300円
- 残留量：300kg

注：
- 毎日の需要は50kgです。
- 売出し価格は購入価格の120%です。
- 残留量は売出し価格の10%です。
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<td>（株）共和電業 ASW-5AMJ6 (角度型) 定格 ±49.93m/s²</td>
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<td>球形変位センサ</td>
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歪型加速度センサのブロック線図
電気（共和電業製ブリッジボックス）のブロック線図
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A-1 PT検査実施計画

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計測位置

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Appendix E

E.1 PSEUDO ACCELERATION SPECTRA OF THE GROUND MOTIONS

Figure E.1 Acceleration spectra for JMA-Kobe ground motion (x-direction).
E.2 Acceleration spectra for JMA-Kobe ground motion (y-direction).

E.3 Acceleration spectra for Takatori ground motion (x-direction).
E.2 PSEUDO VELOCITY SPECTRA OF THE GROUND MOTIONS

Figure E.4  Acceleration spectra for Takatori ground motion (y-direction).

Figure E.5  Pseudo velocity spectra for JMA-Kobe ground motion (x-direction).
Figure E.6  Pseudo velocity spectra for JMA-Kobe ground motion (y-direction)

Figure E.7  Pseudo velocity spectra for Takatori ground motion (x-direction)
E.3 DISPLACEMENT SPECTRA OF THE GROUND MOTIONS

Figure E.9 Displacement spectra for the Kobe ground motion (x-direction).
Figure E.10  Displacement spectra for the Kobe ground motion (y-direction)

Figure E.11  Displacement spectra for the Takatori ground motion (x-direction)
Figure E.12  Displacement spectra for the Takatori ground motion (y-direction)
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