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Parametric Study of ACI Seismic Design Provisions Through Dynamic Analysis of a Reinforced Concrete Intermediate Moment Frame.

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ABSTRACT:

Reinforced concrete moment-resisting frames are structural programs that work to withstand earthquake floor motions via ductile behavior. Their efficiency is main to hinder building crumple and loss of existence throughout a seismic occasion. Seismic building code provisions outline requirements for three classes of strengthened concrete moment-resisting frames: usual second frames, intermediate moment frames, and specific second frames. Large study has been conducted on the efficiency of certain moment-resisting frames for areas of high seismic endeavor such as California. More study is required on the efficiency of intermediate second frames for areas of average seismicity since the present code provisions are situated on past statement and experience. Adapting dynamic evaluation application and functions developed via the Pacific Earthquake Engineering study (PEER) team, a representative concrete intermediate moment frame used to be designed per code provisions and analyzed for targeted floor motions with a view to calculate the chance of give way. A parametric gain knowledge of is used to discover the have an impact on of changes in design traits and building code standards on the seismic response and chance of collapse, particularly the outcome of additional peak and the addition of a powerful column-susceptible beam ratio requirement. The outcome show that the IMF seismic design provisions in ACI 318-08 furnish desirable seismic performance centered on present assessment methodology as gravity design appeared to govern the system. Extra height did not negatively affect seismic efficiency, at the same time the addition of a powerful-column vulnerable-beam ratio did not enormously give a boost to outcome. It is the goal of this

venture to add insight into the design provisions for intermediate moment frames and to make a contribution to the technical base for future standards.

INTRODUCTION: On a day-to-day groundwork, most folks take as a right the ground under their ft. Solid ground is a inspiration that many of us recollect as a one hundred percentage assurance. We power our automobiles, travel to work, play outside, and calm down in our houses with the relief that the floor presents a great groundwork to our everyday life. Nonetheless, the bottom can move and at times transfer violently. Earthquakes or ground vibration can arise from both natural and man-made sources. Essentially the most usual traditional source of an earthquake is action alongside a fault in the earth‟ s crust. Other average advantage reasons incorporate volcanic eruptions or tremendous landslides, which may also be effect of earthquakes. Meanwhile, man-made earthquakes are prompted by such things as underground explosions or mining events. On average, more than one million earthquakes are felt and recorded across the globe in a given yr (Marshak 2007, 207).While most of these occurrences are small and nonthreatening, there are occasional greater earthquakes that can rationale huge damage and loss of lifestyles. In the USA, thirty-nine out of fifty are susceptible to "average or extreme earthquakes" (ATC three-06 1984, 1). It is the task of the structural engineer to design buildings to survive the ground motion precipitated by means of earthquakes. Constructing codes and design necessities released via companies such because the worldwide building Code Council and the American Concrete Institute (ACI) have evolved for the period of the earlier century to support lower loss of life brought on with the aid of a structural fall down during an earthquake. By means of the use of study and past observations, there are files that define the quite a lot of types of structural techniques ready of resisting seismic forces and the design standards wanted for those systems to first-rate continue to exist seismic hobbies. Strengthened concrete second frames are one variety of structural method that's broadly used to resist seismic forces. The design requisites for these frames have been divided into three classes established on the seismic pastime of a building‟ s region: certain moment frames, intermediate moment frames, and traditional moment frames of the ACI e-newsletter constructing

Code standards for Structural Concrete (ACI 318, 2008) outlines the various additional detailing requisites for these frames. Usual second frames are placed in areas of low seismic recreation and follow the common design practices for flexural contributors, columns, and participants in compression and bending. In the meantime, specified second frames are used in areas of excessive seismic endeavour akin to California. These frames have been the focal point of so much study into the design and detailing of concrete individuals with respect to increasing a building‟ s survivability during an earthquake Intermediate moment-resisting frames are used in areas of average seismic undertaking akin to within the Southeastern united states of america. This type of frame design was once added to code necessities after the introduction of distinctive and natural second frames so as to provide recommendations for constructions that don't require the ductility of those utilized in California. The effectiveness of intermediate moment frames continues to be being investigated and up-to-date in building code provisions. The rationale of this study is to add to the expertise base on intermediate second-resisting body efficiency by way of the design and modeling of a average frame centered on current ACI 318 code provisions.Ultimately, the thesis investigated the seismic efficiency of a bolstered concrete intermediate moment-resisting body, and the be trained was once fascinated by 4 predominant areas. First, history study was once carried out on earthquake engineering within the usa and the underlying phenomena concerned with seismic design. This dialogue also integrated background on the development of seismic provisions, typical design tactics utilized by practising engineers, and current research being conducted on efficiency evaluation making use of earthquake simulation. Subsequent, a natural intermediate second frame was design founded on present code provisions and input from the engineering industry. The seismic performance of this body used to be then analyzed and assessed making use of the current assessment methodology being developed with the aid of engineering researchers. In the end, a parametric be taught was carried out to investigate how the body‟ s performance used to be affected by an increase in building top and the addition of a strong-column susceptible-beam ratio.

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Precast Shear Wall important points

Unbonded precast shear walls contain prestressing tendons (strand or bar) which are placed in outsized ducts and are usually not grouted over the size of the tendon (Fig. Three). In reality, the unbonded tendon is the important element that renders unbonded shear wall behaviour advanced to that of monolithic shear walls. For unbonded tendons, there is no stress compatibility between the reinforcement and the adjacent concrete, and the elongation of the tendons is dispensed over the size of the tendon. The resulting uniform stress distribution along tendon length delays yielding of the tendons and increases the quantity of steel that can take part in vigour dissipation by way of yielding. Furthermore, unbonded tendons guard the adjacent concrete from cracking on account that tensile stresses are not transferred from the reinforcement through bond. For that reason, unbonded precast shear walls endure less damage for a given amount of lateral displacement than do reinforced concrete shear walls with bonded reinforcement. The PRESSS (PREcast Seismic Structural methods) study software [2] has shown that unbounded precast shear partitions can be utilized as important lateral

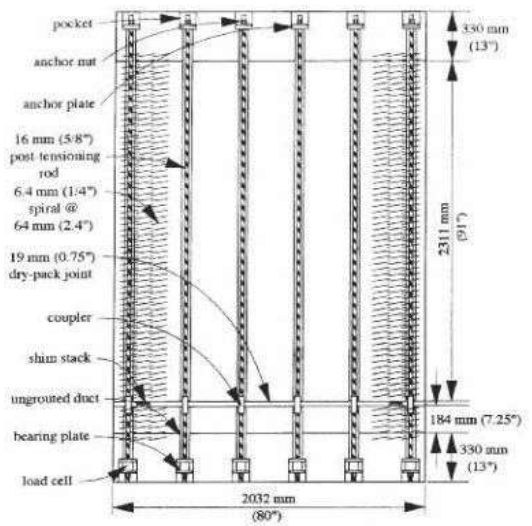

Fig. 3 PTT Wall Specimen Detail

load carrying method within the areas of excessive seismicity. But, there are no design provisions for jointed precast concrete shear wall constructions in mannequin building codes in the us. In latest years, a huge study effort has been expended to satisfy this want. Determine three suggests a plan of a wall specimen presenting unbonded prestressing tendons (PTT) that was once proven as section of the PRESSS software at the national Institute of requisites and technological know-how [3,4]. The tendons comprise excessive-strength bar with couplers placed in ungrouted metallic ducts, and the concrete along the jambs is limited by way of interlocking metal spirals that defend it from compression damage. Short lengths of debonded reinforcement can be utilized to

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increase the efficiency of partitions with bonded reinforcement. This reinforcement element offers the potential of growing the section of the reinforcing bar that undergoes yielding, and it raises the elongation capacity of the reinforcement, the drift capability of the wall, and the quantity of steel that may dissipate energy via inelastic deformation. Figure four indicates a plan of a different wall specimen from the NIST application [4], specimen GSS, which featured grouted splice sleeves to attach the slight steel reinforcing bars, as well as short debonded lengths under the couplers. The bars were wrapped in duct tape over a 7¼-in. Length and covered with oil prior to casting the concrete panel. The GSS specimen (Fig. 4), like the PTT (publish-tensioned tendon) specimen (Fig. Three), had a single horizontal joint between related precast elements. NIST Precast Shear Wall assessments The horizontal joint important points shown in Fig. 3 and four have been a part of a sequence of specimens representing connections at horizontal and vertical joints that have been validated on the national Institute of specifications and technological know-how (NIST) in a research software to represent the seismic performance of connections in precast concrete shear walls

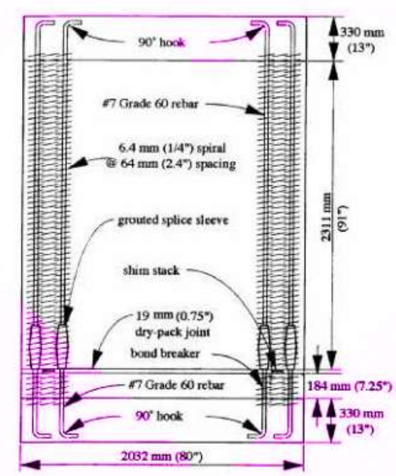

Fig. 4 GSS Wall Specimen Detail

(ref). The 152-mm (6- in.) thick panel specimens have been developed utilizing traditional materials, including concrete with a 34-MPa (5000 psi) compression strength, non-decrease grout, and Grade 60 reinforcing bars (414 MPa), as well as proprietary grouted splice sleeves and put up-tensioning hardware. The horizontal joint specimens in these exams symbolize a component of the prototype wall, together with the panel above the joint and a stub representing the panel beneath the joint. Vertical loads are applied to the top crosshead, the experience and magnitude of which is determined as wanted to outline a vertical compression stress equal to 0.69 MPa (a hundred psi) representing lifeless loading, and a continuity moment equalto 50% of the overturning second on the base of the panel. This last characteristic is vital if a substructure is to be used to appropriately mannequin the minimize component of a taller shear wall. The specimens were tested within the NIST Tri-directional test Facility [5] using cyclic drift histories that simulate seismic motions. The waft historical past used for the tests contains companies of cycles, the pattern of which was repeated until the end of the test, and the peak drift of which used to be accelerated monotonically among successive agencies of cycles. The glide response of specimen PTT to the lateral glide history was once stable over the range of utilized drifts The preliminary response, now not shown in Fig. 5, used to be linear, elastic, but a quickly softening forcedeformation response, i.E., "process yielding", used to be determined as a gap opened at the horizontal joint between panels. Past method yielding, the response was once practically plastic, and remained so over an abundant variety of deformations, that's up to 2.5% go with the flow with out loss of load carrying capability. Unloading behavior used to be also visible to be inelastic even for short excursions past method yielding, resulting in a finite amount of vigour dissipation through hysteresis, although unbonded, put up-tensioned precast shear walls are frequently assumed to show off nonlinear, elastic behavior. The observed in the exams used to be afforded by means of the quick length of the tendons, which used to be dictated through the height of the test specimens, and the distribution of tendons over the size of the wall, alternatively of concentration on the center of the

wall. Beneath these stipulations, the tendons yielded in anxiety and had been deformed to more and more bigger traces with height go with the flow. These stipulations may also be replicated in brief partitions, or in partitions for which tendons are jacked at intermediate features along wall peak, say every two or three flooring. This method is desirable since the wall is stabilized at every jacking station, as a result casting off the necessity for bracing.

SEISMIC DESIGN PARAMETERS

Response Modification Factor, R The response modification factor, R, was introduced in US seismic design practice in 1978 as a way to account for inelastic structural response to earthquake motions [7]. However, the R factor can be traced to the horizontal force factor for allowable stress design, K, first defined two decades earlier [8] for the purpose of differentiating the seismic performance of different structural systems. Initially, only four categories of structural systems were recognized relative to K, namely bearing wall buildings, dual systems, moment-resisting frames, and previously unclassified framing systems. Presently, the NEHRP 2000 provisions recognize 67 different structural systems, and assigns R and Cd factors applicable for strength design. NEHRP specified values for R are intended to reflect past performance and expected amounts of damping, toughness, and overstrength, and there is a recognized need to periodically review these values. Since their introduction, many shortcomings have been expressed concerning code treatment of R factors, primarily regarding the inability of a single-valued parameter to accurately represent all buildings with the same framing type but different (1) plan geometry, (2) height, (3) period, (4) seismic zone and (5) site class (soil type). Consequently, many attempts have been made to interpret the response modification factor, R, in a rational manner with a view towards establishing a rigorous procedure for calculating their magnitudes [9,10,11]. One of the simplest interpretations, and the one most closely compatible with the NEHRP approach was proposed by Uang [10]

CONCLUSIONS After designing the walls and obtaining the pushover response, linear and nonlinear dynamic analyses were conducted using DRAIN-2DX. The linear analyses were needed to define the elastic force demand, VE, needed in Eq. 2 for determining R. The nonlinear analyses were used to determine the inelastic force demand VP , in Eq. 2, as well as the maximum displacement dM needed for comnputing the model ductility µM. The latter of these is used in Eq. (6) to compute µP which is ultimately used to determine Cd from Eq. [3]. Computed values for R (Ranalysis) and Cd for walls of type PTT and SDS=0.8 are shown in Fig. 11 against the values for response modification factor (Rdesign) that were used to design the prototype wall structures. In the present study, all of the prototype walls responded to seismic loading in flexure, with the magnitude of the base moment dictating when the wall yields. As such base moment is more appropriate for defining R than, base shear and R was computed from an expression like the one given in Eq. 2, but using base moments instead of base shears. In Fig. 1, the response to each of the six records in a given Site Class is averaged, as is commonly done in ground motion studies. The magnitude of

Ranalysis is seen to increase in approximate proportion to Rdesign, as expected. However, the relationship is not always linear for all Site Classes. Site Class is seen to have a large impact on Ranalysis, with increasing site deformability (i.e., from A to E) decreasing the ability of the system to generate Ranalysis factors as large as the values assumed in design (Rdesign). From the data above, the optimal R value can be defined as the largest one below which Ranalysis is greater than or equal to Rdesign. Using this criterion, it can be seen that the optimal values for R vary considerably with Site Class. While $R \leq 6.5$ is acceptable for the PTT walls with SDS=0.8 in Site Class A/B, it drops to $R \leq 4.5$ for Site Class C. For all walls with SDS=0.8 in Site Class E, the magnitude of drift complicated the response of the walls. For walls with SDS=0.8 in Site Class E, drifts were so large that, in some cases, they exceeded the deformation capacity limit of 2.5%, which was established on the basis of cross-section mechanics and DRAIN-2DX pushover analyses. This observation was generally the case if Rdesign exceeded 4.5 which indicates that R values for monolithic concrete bearing and shear walls is acceptable for type PTT walls if $SDS \leq 0.8$, especially for rock and hard soil sites. However, for soft soil sites, R should probably not exceed 4.0 for type PTT walls.

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