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DUCTILE CLADDING SYSTEMS FOR SEISMIC DESIGN

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SUMMARY

Architectural cladding offers the potential for passive response modification in building structures designed to resist seismic loadings. The cladding connections can be designed to dissipate energy through inelastic material hysteresis driven by seismically induced interstory drift. When properly designed, precast cladding panels can easily support the added interstory forces and the ductile connectors can stably dissipate energy without compromising connection integrity. From a design point of view, this passive response modification can be used to (a) improve serviceability or (b) reduce cost. Improved serviceability is achieved by optimizing the connectors to maximize energy dissipation and consequently to reduce the peak drift. Reduced cost, as measured by reduced structural weight, is achieved by starting with a baseline structural design and systematically reducing member sizes while adding passive damping in the connectors while maintaining the same baseline performance (as measured by peak drift). Approach (a) is shown to reduce peak displacements by more than 40% from baseline values and has potential for both new and retrofit/rehabilitation applications. Approach (b) is shown to reduce structural weights in new designs by at least 14% for the most conservative design assumptions.

INTRODUCTION

Cladding facades are generally considered as nonstructural elements and are not allowed to contribute any structural function to the building. Instead of trying to structurally isolate and protect facades by avoiding any interaction between the cladding panels and the structure, the present research explores ways to use this interaction to dissipate energy and thereby reduce building response. The basic concept of passive control is to dissipate the input seismic energy, reduce energy dissipation demand in the structural elements, and consequently, minimize potential damage to the structure. It will be shown that, when properly designed, ductile cladding connectors can be used to passively dissipate significant amounts of energy through inelastic hysteretic deformation driven by interstory drift. This concept was identified in earlier studies [Palsson 1982] and preliminary results showed feasible connector designs are capable of dissipating enough energy to reduce peak displacements by nearly 50% [Pinelli 1993]. The present study extends this earlier research to show how ductile cladding connectors can be applied to retrofit a damaged structure or to rehabilitate an existing structure to reduce baseline displacement (improve serviceability). More significantly, for new structures the study also presents a design methodology where the objective is to utilize passive control to allow reduction in structural weight (cost) while not exceeding design displacements or allowable member stresses.

A ductile cladding connection consists of 3 basic parts: (1) a panel anchor, (2) the connector body itself, and (3) another anchor on the building structure. Previous test results [Pinelli 1996] showed that connection anchors are not by themselves capable of dissipating significant energy without damage, so energy dissipation must then occur in the connector body. Inelastic, hysteretic deformation is a well-known mechanism and has been employed in this study. Experimental studies of three different concepts for ductile connector bodies, including tapered flexural tie-back connectors [Pinelli 1992a,b], composite rubber bearing connectors with tapered flexures [Blanchet 1998], and novel torsion tie-back connectors [Khan 1997], have shown promising characteristics for each type. Much of this work is summarized in a recent report [Goodno et. al. 1998].

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Previous design studies using a simple 2D structure [Pinelli 1992] showed that promising levels of seismic response modification (e.g., reductions in peak displacements) can be obtained from a practical design using ductile cladding connections. As such this application is appropriate for rehabilitating or retrofitting existing structures. When considered in new structures, the objective is somewhat different and includes potential cost reductions for the conventional structural system. It will be shown in this paper that passive response modification using ductile architectural cladding connections can lead directly to reduced initial building cost. It should be kept in mind, however, that the ductile cladding design approach described in this paper is a research study and is not fully compliant with existing codes or present guidelines for design of passive damping systems. Nonetheless, it is hoped that these promising results may lead to future revisions in allowable design procedures.

PASSIVE RESPONSE MODIFICATION

Ductile Cladding Design Approach

A cladding connector may be called upon to transmit the following loads (see Fig.1): (a) gravity (vertical) forces from panel weight, (b) normal forces from wind, (c) inplane forces from thermal loads, and (d) inplane/ normal/vertical inertial loads from an earthquake. Since panels usually span one or two stories, inplane forces can also arise from interstory drift. The cladding is not usually required to carry any bending moments so these are not considered (although eccentricities in the connector may introduce significant bending moments in it).

For simplicity and minimum design cost, every effort is made to design a statically determinate connector system. Accepted design practice is to use bearing connectors to support the gravity loads (and normal/inplane loads) at either the two lower or two upper locations and "tie-back" connectors at the other end. The tie-back connectors are flexible in the inplane direction (stiff in the normal direction). This supports wind loads and provides resistance to seismic forces but isolates the panels in the inplane direction from interstory drift and environmental forces. Such an approach is widely employed in the US.

The ductile connectors are assumed to function in every respect as conventional tie-back connectors, EXCEPT they are also allowed to transmit horizontal inplane loads (in the plane of the panel) that arise from interactions between the panel and the supporting structure due to transverse interstory drift. In this respect, the ductile tie-back connectors will allow the cladding to contribute to the interstory shear resistance of the building and at the same time to dissipate energy as a result of this interaction. Such action will introduce inplane shear forces into the cladding panel, and the panel must be capable of supporting such loads. However, by proper design, the connector can also be configured to limit the maximum level of shear force that can be introduced into the panel, thereby protecting not only the panel but also the panel insert and the building attachment.

Use of passive energy dissipation devices suggests that the design criterion might be best formulated in terms of energy dissipation. However, the cladding connectors also add stiffness to the system because of their interstory bracing effect and therefore change the dynamic characteristics of the structure. In turn, the hysteretic properties of the connector are nonlinear and include several passive design parameters, such as initial elastic stiffness, yield force, strain-hardening and overall ductility. In previous studies of such passive devices [Pinelli 1996] an effective design criterion was found to be the ratio of the energy dissipated in the connector to the total input seismic energy for a given "design" earthquake. In other words:

$$ObjectiveFunction = \frac{E_{Connectors}}{E_{Input}} \tag{1}$$

The best design is assumed to be the one that maximizes this ratio. At the same time several additional design constraints must be satisfied: (1) the ductility demand on any of the connections should not exceed an allowable value defined for each particular type of energy dissipator (e.g. from laboratory tests), (2) the connection should be able to satisfy the minimum code requirement regarding strength (e.g. Uniform Building Code 1997, Section 1633.2.4.2); and, (3) the forces induced in the panel by the connections should not exceed the panel capacity.

A key issue is the definition of ductility which is needed to define the maximum (peak and cumulative) allowable deformation in the connector. The simplest definition of ductility is the ratio of maximum monotonic displacement to yield displacement, but this provides only limited information for dynamic systems where load reversals can significantly reduce the maximum ductility. McCabe and Hall [1989] defined the equivalent monotonic plastic ductility μp as follows:

$$\mu_P = \frac{H_t}{f_Y u_Y (2N_f)^{0.4}} \tag{2}$$

where Ht is the total hysteretic energy dissipated in the system during the Nf load reversals. fy and uy are the yield load and yield displacement. This was used to define the connection ductility demand in the present study.

For purely practical reasons, additional constraints should also be added. In general it is unlikely that a given set of connector <u>design</u> properties can be uniquely related to a particular connector geometry, that is, more than a single geometric configuration could yield the same design properties. Therefore, although it will not be considered in this paper, it may be necessary that the connector also satisfy practical criteria related to constructability such as total connector length or certain material thicknesses.

Connector Design Model

A well-designed ductile connector should exhibit an initial elastic deformation, a sharp yield, and a large inelastic deformation. For practical ductile materials (such as mild steel), this behavior can be satisfactorily described by a bi-linear stress-strain diagram, or more simply by an elasto-plastic behavior (zero post-yield stiffness). Such behavior has been observed in laboratory tests of potential connector designs [Pinelli 1996, Blanchet 1998, Goodno 1998], and on this basis a cladding connector design model assuming an elastic-plastic material behavior shown in Figure 2 was assumed. The design variables are taken to be the initial elastic stiffness and the yield force, and the connector maximum dynamic ductility and maximum yield force are constraints. It should be noted that with this type of model, the forces transmitted into the cladding are limited by the connector yield force, and this feature can be used to protect the panels from damage.

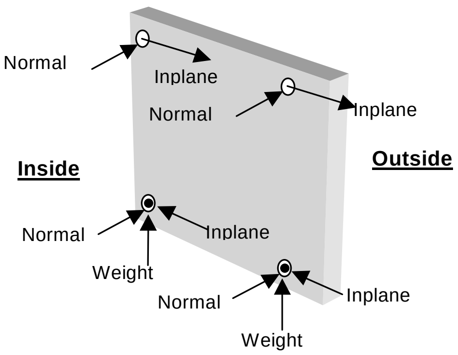

FIG. 1. Schematic Diagram of Cladding Panel Loads

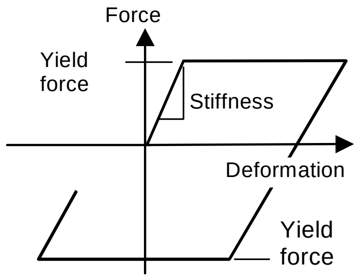

FIG. 2. Assumed Cladding Connector Design Model

DESIGN APPROACH

There are two connection design objectives: (a) reduce peak displacements of the structure and thus improve its serviceability, or (b) reduce needed structural material and thus the structural cost. It should be noted that (a) is appropriate for either new or retrofit designs or perhaps more importantly for rehabilitation to meet performance levels such as those specified in FEMA 273/274 [NEHRP 1997], while (b) is feasible only for new designs.

Design for Serviceability

In this case the design for serviceability objective must incorporate a measure of peak response. The objective function in Eq. 1 does not directly reflect this, but earlier studies [Pinelli 1996] showed that when this objective function is employed, the result is an overall reduction in peak displacements in the structure. The objective function in Eq. 1 must be maximized subject to the constraints noted earlier, the most important of which is on the ductility demand, P, and this can easily be expressed as a simple nonpositive inequality constraint:

$$0 \geq \mu_P - \mu_{LIMIT} \tag{3}$$

where LIMIT is the maximum allowable design ductility as inferred from simple monotonic ductility laboratory tests of a connector design. Additional practical constraints on material dimensions, etc. can also be expressed in similar inequality forms.

The evaluation of the objective function and the constraints must be carried out through a simulation of the building performance, and this raises the question of a suitable "design" earthquake, or more pointedly a "worst-case" disturbance. Needless to say, there is still controversy on how to approach this problem in earthquake engineering. In the present study, the "design" earthquake approach was followed but only a single synthetic design earthquake was selected for the site under consideration. Using a single earthquake for actual design purposes is clearly questionable, but it is more appropriate for a comparative study such as the present.

The nonlinear structural analysis program, DRAIN-2DX [Prakash et.al., 1993] was employed to compute a response time-history of the building for the design earthquake. The nonlinear cladding connectors were modelled using built-in element types in DRAIN-2DX, but the code was modified to track the hysteretic energy dissipation in each element and to accumulate the total input seismic energy so that the objective function could be evaluated. In addition, the connector element type was modified to accumulate the number of load reversals for computation of the dynamic ductility (Eq. 2). Nominal values for nonlinearities in the main structural members were also included in the DRAIN model using the available beam and column element types. The use of DRAIN-2DX was based on prior experience with this code, its relative simplicity (additions to source code are possible), and growing acceptance. On the other hand, DRAIN-2DX is a purely two-dimensional code, but this was felt to be acceptable for an initial study such as the present where complex 3D behavior is not an issue. The DOT code for numerical optimization [Vanderplaats 1993] was employed for the numerical optimization. A simple main program was constructed to implement the flowchart shown in Figure 3. DRAIN-2DX as well as the DOT procedures were all compiled using the DEC Visual Fortran 5.0 development system. Execution was carried out on Pentium II 266 MHz personal computers with 64 Mb or more main memory.

Design for Reduced Structural Cost

Ductile cladding connections can also be used to reduce the seismic demand on the primary structure and thereby allow reduction in the needed structural strengths with a commensurate reduction in the structural weight and cost. In other words, the design strategy is to increase hysteretic damping in the cladding connections, resulting in reduced member forces for design while maintaining a "baseline" structure response level (e.g., no improvement in serviceability).

As before the objective is to determine the optimal design parameters for a ductile cladding system, however, it is also necessary to concurrently consider the design of the primary structural system. Rather than attempt simultaneous design of both the primary structural and passive response reduction systems, a sequential design approach was formulated. This process begins with the synthesis of a primary structure using conventional methods and the result is defined as the "baseline" structure. Next, a passive cladding system is configured and the connector properties are designed as described previously. The result of this step is determination of the peak displacement response reduction, η, that can be achieved for this baseline structure using a ductile cladding system. It is assumed that the resulting structural system is now over-designed because the presence of the passive damping system has reduced the strength demands on the primary structural system. The next step is to reduce the stiffness and strength of the baseline structure until capacity is once again equal to demand. A novel "equivalent" linear redesign process was conceived for this purpose. Response Spectrum Analysis (RSA) is used (in place of the previously employed nonlinear dynamic time history analysis) to define the maximum lateral forces to be used in the structural member selection process. In order to account for the presence of hysteretic damping introduced by the ductile cladding connectors, the assumed modal viscous damping for the RSA is iteratively adjusted to match the peak displacement response with the initial nonlinear time history analysis on the baseline structure.

Figure 4 schematically outlines the redesign process starting with the baseline structural models with and without the optimally designed ductile cladding connectors. In the first iterative loop, RSA is used to compute an estimate of the peak displacement for the design earthquake. To bring the peak RSA displacement value into agreement with the baseline time history nonlinear dynamic analysis, the assumed damping in the RSA is iteratively adjusted until peak displacements are the same at point A. This defines a new "equivalent linear" baseline model.

From point A the next step is to determine the amount of added viscous modal damping to represent the effect of the hysteretic damping provided by the optimal cladding connectors. A second iterative loop ending at point B is used to adjust the added viscous modal damping until the target displacement reduction, η, is achieved. Thus the cladding is assumed to provide no additional lateral stiffness, and the overall viscous modal damping is increased to represent, *in an approximate sense*, the energy dissipation provided by the ductile connectors (the metric being the peak displacements). This is plausible because it has been found that the optimal connector properties generally define a connector with a relatively high initial elastic stiffness but a low or moderate yield force. Such a connector will reach yield level displacements at small lateral response values and thus very quickly begin to develop hysteretic action. As a result, the ductile connectors contribute essentially no additional lateral stiffness to the overall structure model, even though significant initial elastic stiffness values may be identified in the optimal design.

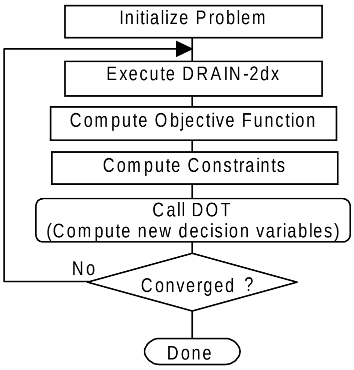

FIG. 3. Numerical Optimization Program Flowchart

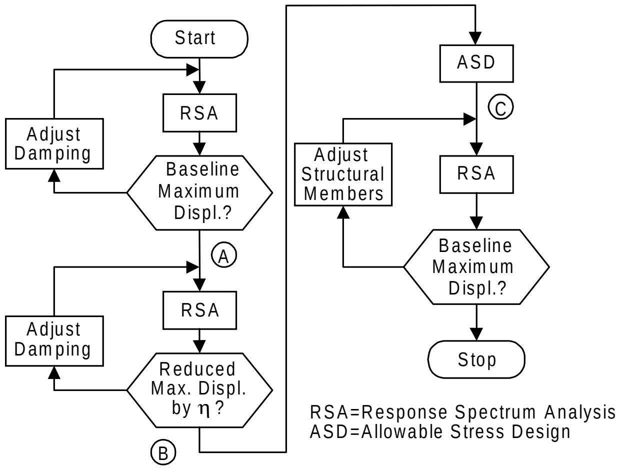

FIG. 4. ASD Steel Redesign Procedure

Next, an assumed frame ductility is used to reduce the elastic response spectrum forces to proper design levels consistent with code-level design forces. Full dead loads (DL) and live loads (LL) are combined with RSA forces (acting + or -), which are reduced to inelastic values by dividing by the assumed ductility factor (d), to form the following load combinations for redesign of the frame:

$$[1.0\ DL + 1.0\ LL\ (+/-\ RSA)/d] \quad (4)$$

Starting from point B the iteration loop shown on the right side represents the structural redesign process to account for the presence of the ductile cladding connectors. The RSA between points A & B establishes the forces to be used in this redesign. These forces will be lower than previously used due to the added damping defined at point B, and therefore the strength and stiffness of the structural system can be decreased by reduction in member sizes. Automated member selection (with constraints on minimum member sizes) is performed based on allowable stress design (ASD) procedures, and the structural system is checked for the specified load combinations. This results in a new structural design with adjusted member sizes at point C.

The last step in the redesign process is to carry out an RSA for the new structural design and to compare the peak displacement response with the original baseline target value. Using the baseline value insures that the modified structural system will be designed to the same serviceability level as the original structural system. A final iteration loop is called for only if the resulting peak response is greater than the target baseline value. In this case, it is necessary to increase the structural system member sizes in order to reduce the peak displacement to the target value, and one or more iterations may be required to accomplish this. On the other hand, if the resulting peak displacement is less than the target value, the design process is stopped. In this case it is assumed that the optimal cladding connector system is capable of a greater improvement in performance (e.g., reduction in peak displacement) than can be offset within code specifications by reductions in member sizes.

BUILDING STUDIES

Baseline Building Structural Model

The study building is located in Northern California and was constructed in the early 1980's. It is a symmetric 20 story steel frame clad with heavy precast concrete panels (Fig. 5). The transverse direction consists of 3 different steel frames: a 3-bay moment resisting frame, a 1-bay braced frame, and a 3-bay braced frame, while the longitudinal direction consists of a 13-bay moment resisting frame system. The floor decks were constructed with 140 mm (51/2 in) thick reinforced concrete. Each exterior bay of both transverse and longitudinal frames supports a 115 mm (41/2 in) thick precast cladding panel.

Due to time and resource limitations in the present study, it was only possible to study the building the longitudinal direction using DRAIN-2DX. Because there are only two 3-bay moment frames containing cladding in the transverse direction, it is unlikely that energy dissipative cladding would be very effective in this direction. The design earthquake was a 50 second synthetic ground acceleration record (475 year, 0.47g peak acceleration) appropriate to the area. The nominal model was based on the full 20 story, 13 bay moment resisting frame and contained 294 nodes, 540 elements, and 840 degrees of freedom. Properties of members used in this model were obtained from as-built structural drawings. One cladding panel per bay was attached to the model. Additional modelling details are in a full report [Goodno 1998].

It was found that compute times exceeding 24 hours were needed to simulate the full 50 second design earthquake using the nominal model, and so a computationally tractable "design model" was created for the numerical optimization process. The design model was a twenty story, one bay moment resisting frame with the overall dimensions identical to one bay of the nominal model. The design model had one cladding panel per bay attached to the frame, and all connection characteristics were similar to those in the nominal model. The design model had 200 degrees of freedom and required less than 3 minutes compute time. In order to make this design model accurately represent the nominal model, the frame properties were adjusted using scaling factors so that the basic dynamic behavior, specifically the periods for the first five modes and the top floor displacements, were in good agreement as shown in Table 1. The beam and column properties were factored until the periods and the top floor displacement of the unclad design model agreed with the unclad nominal model. Similar factors of 13 for the yield force and 17.5 for the initial stiffness were applied to the connector properties.

TABLE 1. Dynamic Behavior of Design and Nominal Models Based on 6 sec. of Ground Acceleration

		Without Cladding Connection		With Cladding Connections	
		Design Model	Nominal Model	Design Model	Nominal Model
Periods (s)	Mode 1	3.302	3.313	2.173	2.176
	Mode 2	1.217	1.235	0.739	0.759
	Mode 3	0.713	0.730	0.416	0.454
	Mode 4	0.507	0.518	0.295	0.323
	Mode 5	0.389	0.398	0.228	0.251
Max Displ mm(in)		272(10.7)	266 (10.5)	232 (9.14)	229 (9.0)

Performance with Ductile Connectors

Numerical optimization to determine optimal connector properties gives only final values or intermediate results (if requested), however, this information provides only a narrow glimpse of the design space as a whole, and it provides little or no insight into the design process itself. As a result, a manual process was also used to develop 2D contour plots of the objective function as a function of the 2 design variables. Both the objective function contour lines and the contours of dynamic ductility demand are superimposed in Figure 6. Two optimal points are indicated by the black dots on the figure. Depending on the different design assumptions employed (which involve different assumed maximum force to be transmitted into the panels and different maximum dynamic ductility demand to be allowed), two different optimal combinations of connector yield force and stiffness values were identified in Figure 6 (labelled as Cases A & B) and are summarized for the nominal model in Table 2.

Both the displacement and period of the structure decrease significantly when the optimal ductile connections are added. The results clearly show the use of ductile cladding connectors results in significant energy dissipation (e.g., see objective function contour values in Fig. 6). The ductile connectors reduce the peak top floor displacement by up to 41% compared to the baseline "conventionally designed" model.

Steel Redesign

The baseline 13 bay moment frame in the longitudinal direction was reanalyzed using the GTSTRUDL structural analysis and design computer program in order to make use of its automated redesign capability. The GTSTRUDL model contained 260 beam elements, 280 column elements, and 260 rectangular panel elements. The lumped mass model for the structure was computed from the self weight of the beams, columns, composite floor deck, precast cladding panels and 9.93 MPa (10 psf) dead load to account for the partitions. A 99.3 MPa (100 psf) reducible live load was also used for the steel redesign sequence. The total structural weight as obtained using GTSTRUDL was 4757 kN (1069 kips).

Following the flowchart in Fig. 4, the target displacement for point A was 585 mm (23.05 inches) at the roof level, and the adjusted damping in the RSA was 4.9% at point A. The next step leading to point B requires selection of one of the optimal ductile connector designs (Cases A&B) in order to define reduction factor, η (from 14-41% from Table 3). Case A, the most conservative with only moderate cladding interaction and a reduction factor of 13.7%, was selected first. The new target peak displacement was then 651 mm (25.6 inches), and after iterative adjustment, a corresponding viscous modal damping of 7.1% was obtained at point B. The RSA also established the lateral forces for the redesign of the baseline structure in the next step.

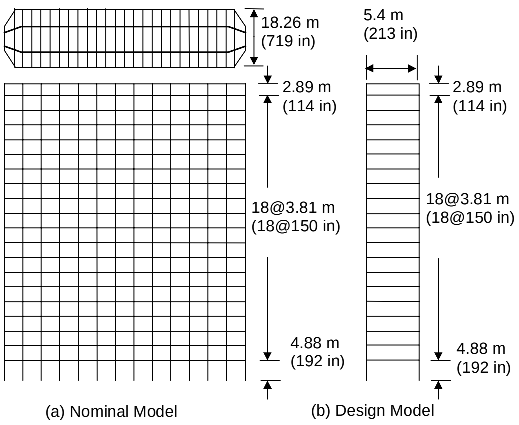

FIG. 5. Building Model Configurations (Not to Scale)

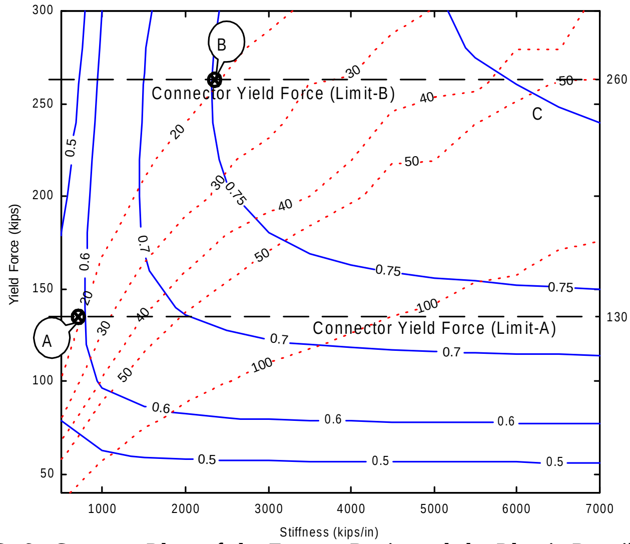

FIG. 6. Contour Plots of the Energy Ratio and the Plastic Ductility Demand for Design Model (Note: 1 kip = 4.48 kN, 1 kip/inch = 175 kN/m)

TABLE 2: Optimal Connector Design and Performance Values for Nominal Model

Case	Yield Force kN (kips)	Initial Stiffness MN/m (kips/in)	Dyn. Duct. Demand	Fundamental Period (sec)	Max. Top Disp. mm (in)	Reduction Factor η
Base	--	--	--	3.31	585 (23.0)	0%
A	44.5 (10)	6.0 (34.0)	20	2.63	505 (19.9)	13.7%
B	89 (20)	24.0 (137.0)	20	1.85	419 (16.5)	28.5%

Next, the target peak displacement was set back to the baseline value of 585 mm (23.05 inches) to insure that the modified structural system was redesigned to the same serviceability level as the original structure. Member selection and design check procedures according to the provisions of ASD89 were automatically performed in GTSTRUDL. It should be noted that ASD89 was used to more accurately approximate the original building structural design which was based on the 1982 Uniform Building Code. The minimum member sizes for the beams had to be restricted to practical values. The weight of the resulting structural frame was computed by GTSTRUDL to be 4154 kN (934.0 kips) which represented a 12.6% reduction compared to the baseline design. Table 3 compares the distribution of weight between the beams and the columns for this design, and it compares the overall structural performance as measured by base shear, overturning moment and period. Not surprisingly, the increased flexibility in the redesigned structure lengthens the fundamental period and yields lower values for the base shear and overturning moment.

The same redesign procedure was carried out for Case B from Table 2 which defines a more aggressive use of ductile cladding (more interaction). The target peak displacement reduction was 28.5% from Table 2 and the resulting weight reduction for the steel redesign increased to 16.8% compared to the baseline. The behavior for Case B was generally similar to Case A in all other ways [Goodno 1998].

CONCLUSIONS AND RECOMMENDATIONS

In the present study, isolating tie-back connectors were replaced with ductile connectors, and numerical optimization methods were used to determine the optimal design properties. Practical constraints on maximum force and initial stiffness as well as on maximum dynamic ductility demand (to insure a reliable service lifetime) were added. The design study considered two different (and complementary) objectives: (1) use ductile connectors to provide an additional margin of performance (and safety) for the baseline building, and (2) use ductile connectors to reduce structural demands and allow reduction of the steel member sizes in order to provide essentially the same baseline level of performance, but in this case using less steel in the primary structure. In order to verify the application of ductile cladding connections, a selected building was evaluated in its longitudinal direction only, and for objective #1, up to 41% reduction in peak displacements was achieved. For objective #2 which is applicable only to new designs, results showed from 14-17% reduction in structural steel for the longitudinal frame system while maintaining the same level of performance as in the baseline building.

TABLE 3 : Summary of GTSTRUDL Design Results for Steel Redesign

Case	Damping (%)	Beam Weight kN (kips)	Column Weight kN (kips)	Steel Weight kN (kips)	Total Base Shear MN (kips)	Overturning Moment MN-m (kip-ft)	Fund. Period (sec)
Baseline	4.9	1792 (402.8)	2966 (666.6)	4757 (1069)	15.45 (3473)	954.7 (704,072)	3.31
Case A	7.1	308.0 (-30.8%)	626.0 (-6.5%)	934 (-14.5%)	10.93 (2457)	543.7 (400,937)	3.95

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