by Eric Elsesser

7.1 INTRODUCTION

Design of any building is a challenge for architects and engineers, and the challenge is made more complex by providing for earthquake resistance. During the past 100 years, seismic design philosophy and details have progressed from simply considering earthquakes to be the same as wind loads, to a sophisticated understanding of the phenomenon of the earthshaking that induces a building response.

This chapter covers the 100-year history of seismic structural systems as developed by engineers and architects, ranging from simple to sophisticated solutions. Basic structural behavior is outlined; guidance for selecting a good structural system is suggested, and the following issues are discussed:

- Scale and size of buildings and structural components
- The impact of building configuration
- Force verses energy
- Drift or movement
- Structural mechanisms (passive to active)
- Costs and post-earthquake repair costs

7.2 A BRIEF SUMMARY OF 100 YEARS OF STRUCTURAL SEISMIC DESIGN

Seismic exposure has extended over many centuries, but systematic seismic design has occurred only over the past 100 years, especially in California since the 1906 San Francisco earthquake.

A group of thoughtful and creative engineers, responding to the observed damage in the 1906 earthquake, started to study, conceive, and design a progression of structural solutions to solve the earthquake response problem. This creative work has extended over a 100-year period, and continues today. A brief progression of key milestones in this seismic design history follows:

- Initial seismic designs for buildings were based on wind loads, using static force concepts. This approach started in the late 1800s and lasted to the mid-1900s.
- After the 1906 San Francisco earthquake, concepts of building dynamic response gained interest, and in the early 1930s, initial studies of structural dynamics with analysis and models were initiated at Stanford University. This approach ultimately led to a design approach that acknowledged the importance of building periods and dynamic rather than static design concepts.
- Dynamic design concepts were enhanced by the acceleration spectra method used for design as developed by Professor Housner at the California Institute of Technology (See Chapter 4, Section 4.5.3).
- While analysis methods were being developed, engineers needed additional knowledge about nonlinear behavior of structural components. Substantial testing of materials and connection assemblies to justify actual behavior were undertaken from 1950 through 1990 at numerous universities (University of California, Berkeley; University of Illinois; University of Michigan; University of Texas; etc.).
- Since 1980 to the present, sophisticated computer analysis programs have been and continue to be developed to facilitate design of complex structural systems and the study of nonlinear behavior.

In the past 70 years substantial change and progress have taken place, not only in California but also over the entire United States, so that concepts and systems can now be utilized that previously could only be dreamt about.

7.3 HISTORIC AND CURRENT STRUCTURAL–SEISMIC SYSTEMS

7.3.1 Early Structural Systems–Pre-1906 San Francisco Earthquake

San Francisco in 1906 had a varied building stock with a few basic structural systems widely represented. Almost all common residential buildings were of light-frame wood construction, and most performed well in the earthquake, except for those on poor, weak soils or those with unbraced lower story walls. Most small- to medium-sized business buildings (about five to six stories in height) were constructed with brick masonry-bearing walls, using wood-framed floors and roofs. These buildings had a variable performance, with upper stories experiencing partial collapse and masonry walls typically showing shear cracks to varying degrees. Tall buildings, constructed during the previous 10 to 15 years (prior to 1906), utilized a steel frame to support gravity loads and provided unreinforced brick/stone perimeter walls which served to provide lateral load resistance. These buildings generally performed well during the earthquake. Most buildings, when subjected to the firestorm after the earthquake, did not do well.

The general conclusion following the 1906 earthquake was that a steel-framed building designed to support gravity loads and surrounded with well-proportioned and anchored brick walls to resist earthquake forces was a superior structural system, and it was commonly adopted by the design profession.

7.3.2 The Early Years (1906 – 1940)

Immediately after the 1906 earthquake, when reconstruction and new buildings became essential, a variety of new structural concepts were adopted. Brick masonry infill walls with some reinforcing were used, and steel frames were designed to carry lateral loads using one of the following ideas: knee bracing, belt trusses at floors to limit drift, rigid-frame moment connections using column wind-gussets, or top and bottom girder flange connections to columns.

As concrete construction became popular after 1910, concrete moment-frame buildings together with shear walls, emerged for industrial and lower height commercial buildings. Concrete slowly replaced brick as a structural cladding after 1930, and buildings commonly used a light steel frame for floor support with a complete perimeter concrete wall system for lateral loads.

7.3.3 The Middle Years (1945 – 1960)

Immediately after World War II, construction of large projects started again. New ideas were common, and some refinement of framing systems for tall buildings was proposed and adopted.

Expressive structural systems were studied and used, but they were usually covered from view with conventional exterior finishes.

The transition from riveted connections to high-strength bolted joints occurred in the 1950s. By 1960, another steel connection change was starting to occur; girder flanges welded directly to columns to create moment frame connections. Because engineers initially did not trust the limited use of moment frames, structural designs were conservative, with substantial redundancy created by utilizing complete moment-frame action on each framing grid, in each direction.

7.3.4 The Mature Years (1960 – 1985)

The 25-year period from 1960 to 1985 represents the "mature years", in that substantial projects were completed using the concepts of either ductile moment frames or concrete shear walls.

The structural engineering profession accepted the validity of 1) ductile concrete-moment frames, 2) ductile shear walls, or 3) ductile welded steel-moment frames as the primary structural system for resisting lateral loads. The primary design activity became optimization of the system or, in other words, how few structural elements would satisfy the minimum requirements of the building codes. Substantial connection tests were carried out at university laboratories to justify this design approach.

7.3.5 The Creative Years (1985 – 2000)

After the damage caused by the 1989 Loma Prieta earthquake (San Francisco Bay Area) and the 1994 Northridge earthquake (Los Angeles), the structural engineering profession began to ask itself about actual earthquake performance. Would real performance differ from the solution obtained by simple compliance with the building code? This investigative process defined many issues, and one of the most important was the dissipation of seismic energy by the building structure. The pursuit of this issue led engineers to the consideration of dual systems and seismic isolation to limit lateral displacement.

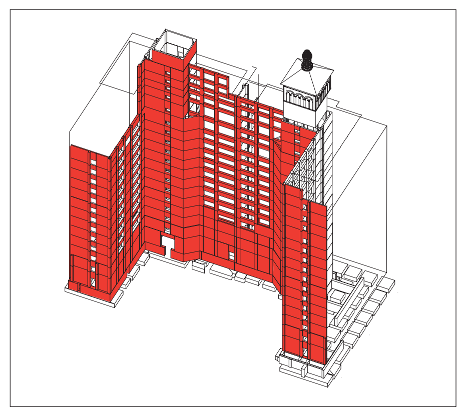
Several significant solutions have been developed using the dual-system concept with stable cyclic seismic behavior:

1. **Dual–system of steel moment frames and eccentric braced frames.** The more rigid eccentric brace provides primary stable cyclic behavior, while the moment frame provides good flexural behavior as a back-up system.

2. **The dual-system steel-moment frame and passive seismic dampers** provide high damping, which significantly reduces the seismic loads imparted to the moment frame.

3. **Unbonded steel braces** with the brace providing stable tension-compression behavior, a significant improvement over the conventional braced frame.

4. **Coupled 3-part systems with moment frames, links, and shear walls** to provide a progressive resistance system in which the resistance progresses from the most rigid system to the more ductile-flexible system.



5. **Seismic isolation,** developed in the early 1980s, is a completely different and reliable concept, in which the building structure is supported on isolation bearings and is effectively separated from the ground, significantly reducing seismic response.

Each of these systems is part of an overall framing concept. These dual and stable mechanisms represent the current search for reliable seismic performance.

7.4 BACKGROUND AND PROGRESSION OF STRUCTURAL–SEISMIC CONCEPTS

The progression of seismic systems selected by structural engineers has resulted from three factors:

1. Study of Past Earthquakes

   Learning from past earthquake performance: Successful seismic structural systems continue to be used; unsuccessful systems are eventually abandoned. New and better ideas frequently flow from observed earthquake damage.

2. Research Data

   New ideas for structural concepts are frequently developed jointly by design engineers and university research laboratories. These systems are physically tested and analytically studied.

3. Building Codes

   Finally, structural systems, that are listed in building codes eventually are used by many engineers as "approved". The problem with code concepts, in these times of rapidly changing systems, is that codes are created about 5 to 10 years after new ideas are developed, so that codes may no longer be current or at the cutting edge of new thinking; overly specific codes may tend to stifle and delay new ideas.

7.4.1 Development of Seismic Resisting Systems

Over a 100-year period, seismic resisting systems have developed substantially. The use of San Francisco buildings as a typical measure of the evolution provides a good summary of the past and present, and some indication of the future.

A summary of individual buildings gives a clue as to thinking. The plot of systems (Figure 7-1) connects the concepts and indicates the progression of ideas.

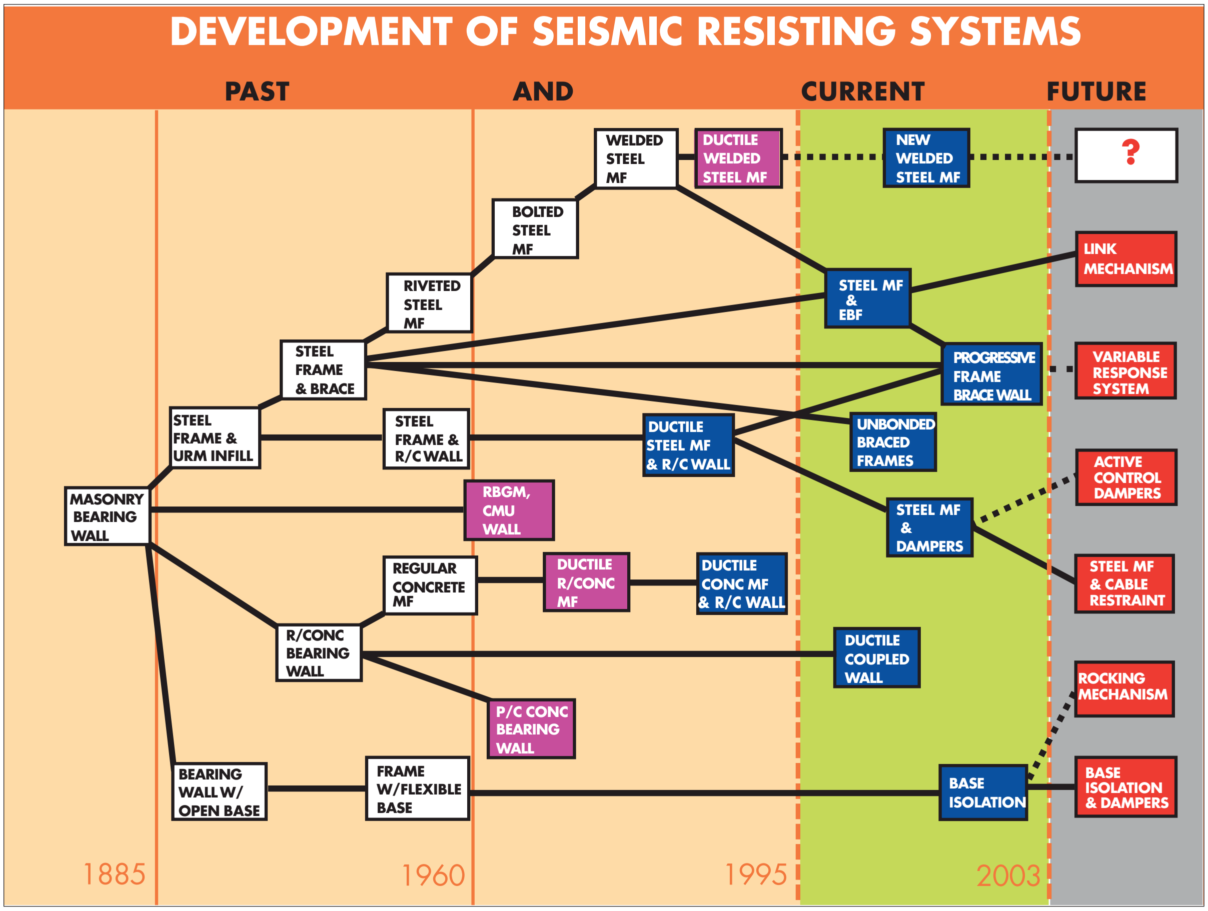


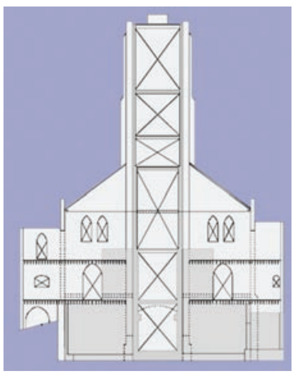
Figure 7-1: Development of Seismic Resisting Systems.

7.4.2 Pictorial History of Seismic Systems

The following pages provide a visual history of key features in the evolution of seismic systems developed and utilized in San Francisco and other Western regions of high seismicity.

Early Seismic Structural Systems **pre-1906 San Francisco earthquake**

photo: historic

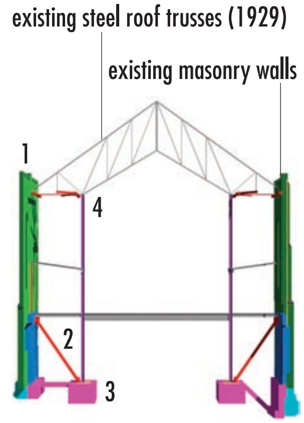


Old St. Mary's Church San Francisco
1869 - 1874

Strong tapered masonry tower,
with nave built with unreinforced masonry buttressed walls. Wood roof of nave burnt after earthquake, but walls all stood without failure.

Good masonry construction with buttressed walls, which apparently resisted seismic loads.

Architect: Crane & England



Nave Cross-Section

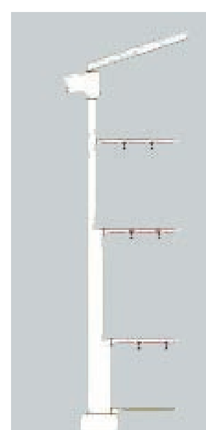
1 Center cored wall reinforcing
2 Basement wall buttress struts
3 New foundation
4 Roof diaphragm bracing

Old U.S. Mint San Francisco
1869 - 1874

A massive masonry-bearing wall building with many thick interlocked walls. The walls benefited from continuous horizontal interlocked brick and a heavy horizontal diagonal rod bracing system at the attic level. The building performed well in the1906 earthquake.

Architect: Alfred B. Mullet

Good masonry reinforcing provided continuity, and steel tension anchor rods provided an effective roof diaphragm.



Early Seismic Structural Systems **pre-1906 San Francisco Earthquake**

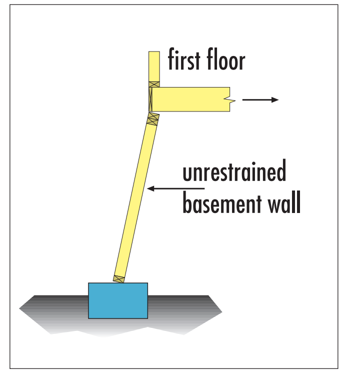
 photo: unknown	**Light Timber Frame San Francisco 1880 - 1930** Wood framing provided good seismic performance, provided that the foundation system is stable and the lowest level is shear resistant and anchored to a strong foundation (not unreinforced brick). This was not the situation with this building. Architect/Builder: Unkown	
	**U.S. Court of Appeals San Francisco 1902 - 1905/1931** A U-shaped, steel-braced frame building with small bays and substantial masonry infill. Concept worked well in 1906 earthquake, even though adjacent to a soft bay-mud soil type. An addition was added in the 1930s. Architect: James Knox Taylor Architect (Addition) George Kelham Engineer: (Addition) H.J. Brunnier Architect/Engineer (Upgrade 1996): Skidmore, Owings and Merrill	This structure was upgraded and restored utilizing a base-isolation solution, 1996.  base-isolation upgrade
	**Mills Building San Francisco 1891 and 1909** Strong form, good steel framework, and good unreinforced masonry infill walls all combined for a seismic resistant building. Architect: Burnham & Root, Willis Polk	Building was the tallest in San Francisco when it was constructed and demonstrated the seismic strength of steel working in conjunction with masonry infill.

photo: unknown

Ferry Building San Francisco
1895 - 1903

Steel-framed building with masonry perimeter, large arched roof truss probably acted as an energy absorbing roof diaphragm spring. Tower was rod braced for wind loads.

The project has been recently seismically upgraded.

Architect: A. Page Brown / E. Pyle
Seismic Upgrade Architect: SMWM
Seismic Upgrade Structural Engineer: Rutherford/Chekene

Steel frame and details demonstrated its ability to absorb seismic energy.

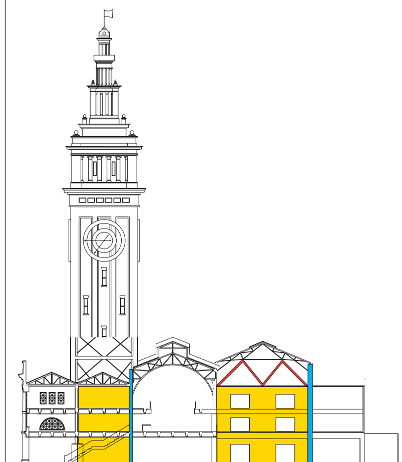
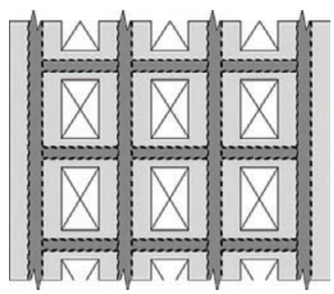


photo: Presidio Army Library

St. Francis Hotel San Francisco
1904 - 1905

Initial building was U-shaped steel frame with URM infill walls, which performed well during 1906 earthquake. A successful seismic design with structural steel frame and masonry infill.

Architect: Bliss & Faville



partial elevation

Flood Building San Francisco
1904

12-story wedge-shaped office building survived the 1906 earthquake and fire.

Architect: Albert Pissis

Another major steel frame that provided vertical support surrounded by brick masonry, which added energy absorption. This combination saved the monumental building in the 1906 earthquake.

Early Seismic Structural Systems **1906 San Francisco Earthquake**

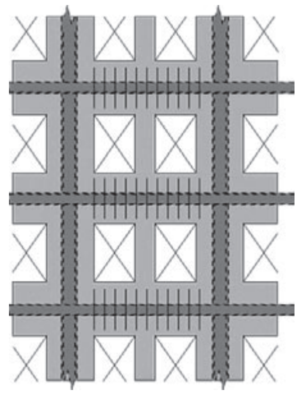
 photo: Stanford University Archives	**Memorial Church** **Stanford University** 1906 earthquake damage shown. Additional damage in 1989. Seismic upgrade after 1906 and 1990. Architect: Charles Coolidge & Clinton Day Engineer for 1990 upgrade: Degenkolb Engineers	Timber and masonry memorial church. Large unrestrained walls collapsed.
 photo: Stanford University Archives	**Library Building** **Stanford University** Serious damage, building demolished after 1906 earthquake. Architect: Unknown	Steel-framed dome & drum with rigid masonry drum. Masonry drum was too flexible and not restrained by masonry walls.
 photo: San Francisco Library	**Old City Hall** **San Francisco** Serious damage; building demolished after 1906. Architect: Augusta Laver	Steel-framed building with unreinforced masonry walls. Steel-framed dome & braced drum with rigid masonry cladding. Masonry drum was not restrained against "ovaling" during the earthquake and caused the brick cladding to separate from the steel frame.
 photo: San Francisco Library	**Downtown, Union Square** **San Francisco** Buildings survived after 1906 and continue to function today Architect: Various Architects	Union Square-steel framed buildings with masonry walls. Bare steel buildings were under construction during the 1906 earthquake. Good performance.

Royal Globe Insurance Building
San Francisco 1907

A post-1906 earthquake building constructed with steel frame and reinforced brick perimeter infill. This was the first reinforced masonry facade in San Francisco after 1906.

Architect: Howells and Stokes
Engineer: Purdy and Henderson

Brick was reinforced apparently to prevent cracking and falling of URM.



City Hall San Francisco
1913 - 1915

A replacement for the original monumental City Hall which failed in 1906. The new City Hall was rapidly constructed to coincide with the 1915 Pan Pacific Exposition. Steel-framing connections were designed for gravity loads without moment connections to facilitate the rapid erection of steel. Virtually the entire seismic resistance was provided by infill perimeter masonry walls. The engineer also believed in a seismic softstory as a means of protecting the building, relying solely on the massive masonry perimeter walls with simple connections between embedded steel columns and girders.

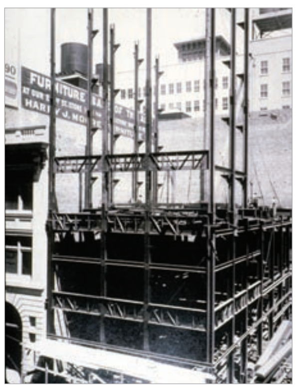
Architect: Bakewell & Brown
Engineer: C.H. Snyder
Reconstruction Engineer:
Forell/Elsesser Engineers

City Hall suffered significantly in the 1989 earthquake and was retrofitted with a real “soft-story” using 530 elastomeric isolators at the base of structure.

Shreve Factory San Francisco
1908

An early reinforced concrete frame building, slender columns, minimum girders–a hazardous condition. Building was retrofitted in 1982 with new concrete shearwalls to protect the weak concrete frame.

Retrofit Architect: MBT Architecture
Engineer: Forell/Elsesser

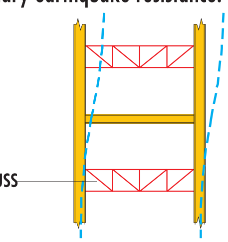


Typical Office Building San Francisco

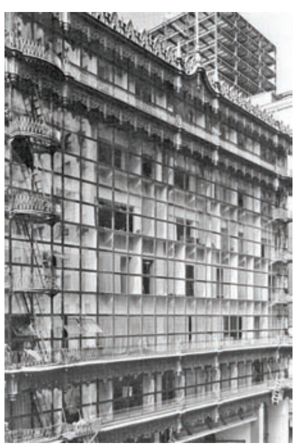
A typical steel frame of the 1920s with belt-trusses used to limit lateral drift to the columns. The entire perimeter was encased in reinforced concrete for stiffness and fireproofing.

Architect: unknown

This system relied on the steel for seismic load resistance, and utilized the concrete walls for primary earthquake resistance.



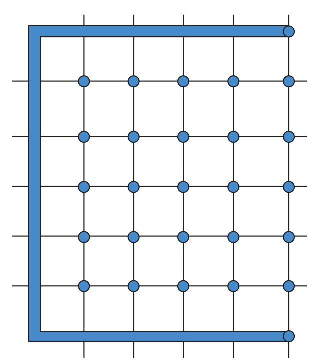
A classic industrial-style concrete building designed to resist earthquakes by a combination of shear walls and slab-column frames, and to provide fire resistance.



Hallidie Building San Francisco
1917

Concrete construction had been in general practice for about 10 years. This was an early reinforced concrete flat slab building, noted for its famous glass facade facing the street. This is a 3-sided perimeter shear-wall building with some moment-frame action between slab and columns.

Architect: Willis Polk



floor plan

photo: H.J. Brunnier

Standard Oil Building San Francisco 1922 / 1948

A large steel frame designed with seismic knee braces, this U-shaped building was tall for the time, 25 stories. Cladding was infill URM, which provided additional drift

Architect: George Kelham
Engineer: H.J. Brunnier

A continuation of the 1906 framing concept with added steel bracing.

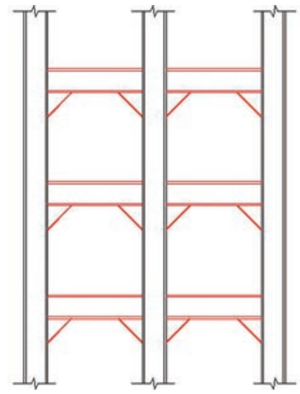


photo: H. J. Brunnier

Shell Oil Building San Francisco 1929

A tall, slender 28-story steel frame structure, with steel wind gusset plate connections for lateral load resistance, together with concrete and unreinforced infill.

Architect: George Kelham
Engineer: H.J. Brunnier

A good example of wind-gusset plate joints for primary lateral resistance. The gussets provided enhanced moment-joint connections.

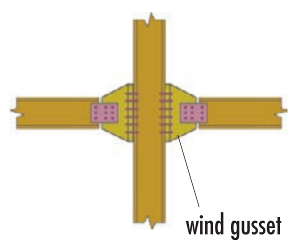


photo: H.J. Brunnier

Russ Building San Francisco 1927

The tallest building in San Francisco at the time, with full riveted moment connections, encased in reinforced concrete and URM facing walls.

Architect: George Kelham
Engineer: H.J. Brunnier

A continuation of post-1906 construction but with steel acting together with reinforced concrete infill walls to resist seismic forces.

photo: H.J. Brunnier

Mills Tower Addition San Francisco 1931

An addition to the successful 1891 original Mills Building. This 1931 addition utilized steel-moment frames created by top and bottom T-shaped flange plates with riveted connections. This type of moment joint was an early use of top and bottom tee-plates, which became standard for 30 years.

Architect: Willis Polk
Engineer: C.H. Snyder

An early use of full moment connections to provide the complete seismic capacity.

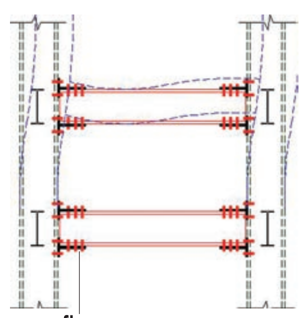


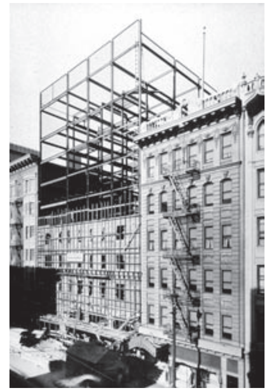
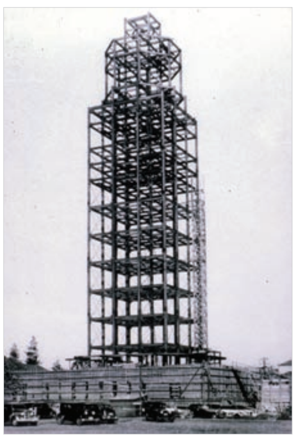
 photo: Bethlehem Steel	**Typical Apartment Building San Francisco 1920 - 1930** A slender steel framework designed to support the concrete floor loads. All lateral loads are resisted by the reinforced concrete perimeter walls. Architect: unknown	 photo: Bethlehem Steel
 photo: Bethlehem Steel	**Hoover Tower Stanford 1938** A minimum steel frame designed as initial supports for floors (similar to apartment buildings), full lateral seismic loads are resisted by a heavy reinforced concrete perimeter wall. Architect: Bakewell & Brown	This building demonstrated the ultimate use of the light steel frame with concrete perimeter walls, acting to resist all wind and earthquake loads.
	**US Appraisers Building San Francisco 1940** A 12-story office building designed with a heavy perimeter steel moment frame utilizing belt-trusses at each floor, and heavily reinforced concrete perimeter walls for additional lateral capacity. An unreduced base shear of 0.08W was used. Architect/Engineer: U.S. Government	

photo: Degenkolb

Park Merced Apartments
San Francisco 1947

Cast-in-place L-shaped, 12-story concrete residential towers utilizing perimeter shear walls.

No steel frame.

Architect: L.S. Shultz
Engineer: John Gould

The beginning of full reinforced concrete-bearing and shear-wall buildings.

photo: Degenkolb

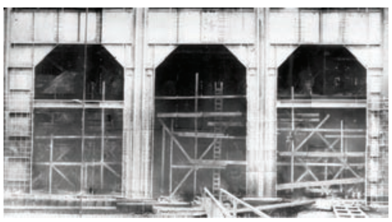


photo: San Francisco Library

Standard Oil Building Addition
San Francisco 1948

The second steel-frame addition required a substantial fabricated plate moment frame to accommodate the seismic forces created by the tall first story. The balance of the framework was a riveted moment frame utilizing top and bottom flange connections.

Architect: George Kelham
Engineer: H.J. Brunnier

photo: American Bridge

Equitable Building
San Francisco 1955

A unique steel moment-frame design utilizing the combined gravity and lateral load moment diaphragms to size and form the girders. Girder splices are at mid-span for shear only. The entire perimeter is encased in a substantial reinforced concrete wall. A lost opportunity to express the structure of a tall building, but a stout seismic solution.

Architect: Wilber Peugh
Engineer: Keilberg & Paquette

photo: American Bridge

photo: Bethlehem Steel

Bethlehem Steel Building
San Francisco 1958

A 14-story steel 2-way moment-frame structure without supplemental systems except for concrete fireproofing. All column-girder construction utilized a bottom flange girder haunch for drift control. Exterior columns offset from girders required a torsion box connection.

Architect: Welton Becket
Engineer: J.A. Blume

One of the last all-riveted structures.

Hancock Building
San Francisco 1959

A 14-story reinforced concrete shear-wall structure with central core and perimeter pierced walls. An elegant reinforced concrete building with a symmetrical configuration.

Architect/Engineer:
Skidmore, Owings & Merrill

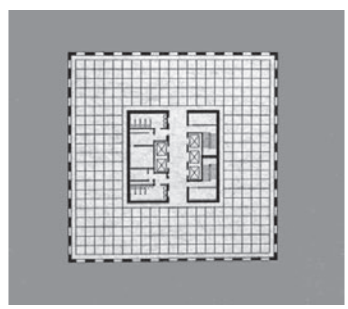


photo: Gould and Degenkolb

American President Lines
San Francisco 1960

A 22-story all-steel moment-frame building with large top and bottom T-section flange connections utilizing high-strength bolts. Joints were tested at the University of California, Berkeley.c

Architect: Anshen + Allen
Engineer: Gould & Degenkolb

A well-conceived steel frame with 2 directional frames on all grid lines.

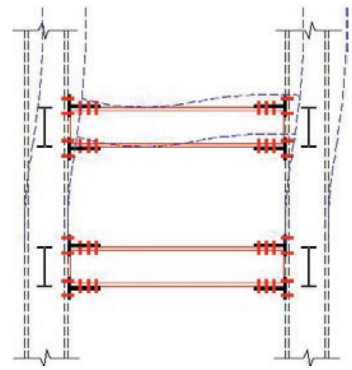


photo: H. J. Brunnier

Zellerbach Building
San Francisco 1960

A monumental 19-story steel-frame building with 60-foot clear span for the office wing, which is a 2-way moment-frame structure, and an adjacent connected braced service tower.

Architects: Hertzka & Knowles, SOM
Engineer: H.J. Brunnier

A steel moment-frame statement for the clear span offices, but with concrete walls around the braced service tower to add seismic strength and stiffness.

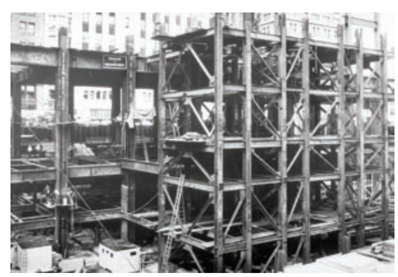


photo: H. J. Brunnier

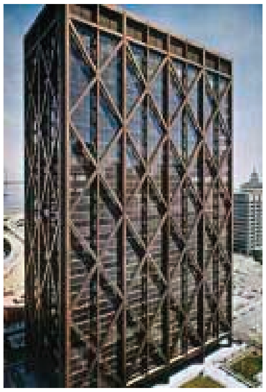
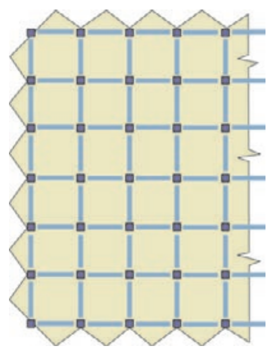
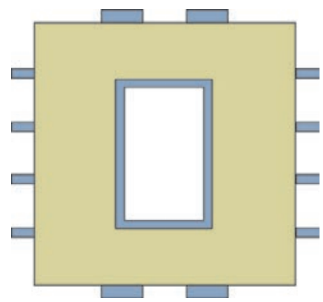
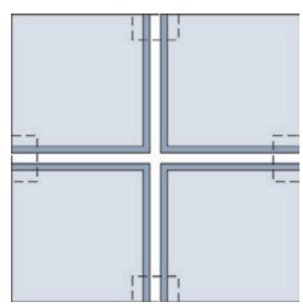
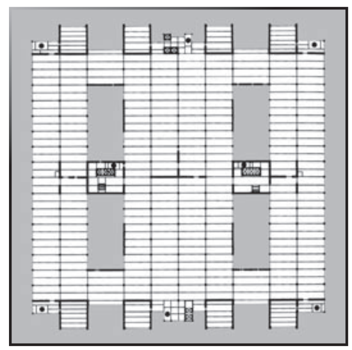
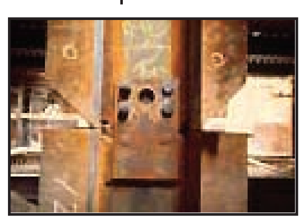
 Photo: Bethlehem Steel	**Alcoa Building San Francisco 1964** A building where a portion of the seismic bracing is expressed. The complete seismic system is dual: 1) the perimeter diagonal bracing, and 2) an interior moment-resisting 2-way frame. Architect/Engineer: Skidmore, Owings & Merrill	An elegant solution to express the structural seismic needs of the building.  Photo: Bethlehem Steel
 Photo: H.J. Brunnier	**Bank of America Building San Francisco 1968** At 50 stories, the tallest, full-plate floor building in San Francisco, utilizing a 2-way grid of steel-moment frames with box column welded connections. Architect: Skidmore, Owings & Merrill and Pietro Belluschi Engineer: H.J. Brunnier	One of the last major 2-way moment-frame grid structures. 
	**Transamerica Tower San Francisco 1972** The tallest structure in San Francisco with its pyramid steel-framed form. A sloping moment frame above is supported on triangular pyramids towards the base. These pyramids transition to vertical frame columns at the lowest level. Architect: William Pereira Engineer: Chin and Hensolt	A unique and special structure designed as a "monument" for the Transamerica Corporation. The figure shows the base structure for columns.  Photo: Bethlehem Steel

 photo: H.J. Brunnier	**Golden Gateway Towers** **San Francisco 1960** 25-story reinforced concrete dual shear-wall and moment-frame tower supported on massive drilled pier foundations. The first concrete tower blocks with low floor-to-floor elevations. Architect: Skidmore, Owings & Merrill / Wurster Bernardi & Emmons / Demars & Reay Engineer: H.J. Brunnier	The dual concrete shear walls and moment frames represented a new direction for San Francisco's high-rise buildings. This project was an early, ductile concrete moment frame.
	**Summit Apartment Building** **San Francisco 1968** An expressive tower reflecting the seismic and gravity loads in the building form. Post-tension floor plates. Architect: Neil Smith / Claude Oakland Engineer: S. Medwadowski	A landmark concrete structure on top of Russian Hill.  Floor Plan
	**New St. Mary's Cathedral** **San Francisco 1971** A unique reinforced concrete shell form designed by Pier Luigi Nervi with an expressive structural form. A single structure-form solution. Architect/Engineer (for shell): Pier Luigi Nervi	A special hyperbolic shell solution that blends form with structure.  Roof Plan

	**Pacific Medical Center** **San Francisco 1970** A tall, interstitial-space building designed as a moment frame and encased in a reinforced concrete shear-wall system. Architect: SMP Engineer: Pregnoff/Matheu	One of the "hospital systems" buildings developed in the 1970s.  photo: SMP
 photo: BSD	**Loma Linda Hospital** **Loma Linda 1975** A hospital system building with a dual steel moment-frame/ concrete shear-wall seismic solution designed to resist real ground motions and minimize damage. Architect: Building System Development and Stone, Maraccini and Patterson Engineer: Rutherford & Chekene	 Plan
 photo: Degenkolb	**UCSF Hospital** **San Francisco 1978** A substantial seismic demand for this site dictated steel-plated shear walls to provide adequate seismic strength. Architect: Anshen + Allen Engineer: Degenkolb	 photo: Degenkolb interior steel plate shear walls  boundary column splice

Seismic Structural Systems **The Mature Years (1960 - 1985)**

photo: Skilling, et al

Bank of America Center
Seattle 1965

50-story, 850,000 sf-in-area bank and office tower utilized a shear wall concept with 4 exterior walls designed with vierendeel trusses spanning to corner columns. At the 6th floor level, earthquake/wind loads are transferred to steel plate core walls.

Architect: NBBJ
Engineer: Skilling, et al.

A unique dual-perimeter and interior structural solution.

photo: Skilling, et al.

photo: Bethlehem Steel

Metropolitan Life Building
San Francisco
1973

A two-directional welded steel moment frame using shallow girder haunches at columns for drift control. Box columns for 2 way frame action.

Architect/Engineer:
Skidmore, Owings & Merrill

photo: Bethlehem Steel

Marathon Plaza
San Francisco 1985

A dual concrete seismic system with cast-in-place perimeter ductile moment frames and a cast-in-place shear-wall core. Precast panels clad the exterior, forming the exterior surface of the perimeter girder.

Architect: Whistler Patri
Engineer: Robinson Meier Juilly

A common, economical dual-system seismic solution, interior walls, exterior moment frame.

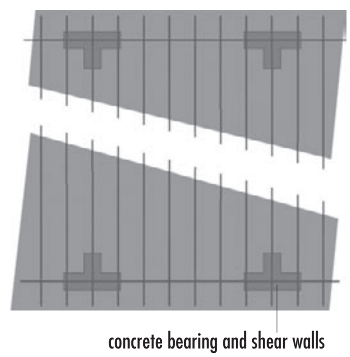
	**Pacific Mutual San Francisco 1979** A perimeter welded steel-moment frame solution with light-weight GFRC cladding to minimize seismic mass. Architect: William Pereira Engineer: Chin & Hensolt	The use of deep but light-weight column sections to force yielding to occur in the girder sections. 
	**Tomales High School Tomales 1980** A school "systems" building with custom-designed concrete cantilevered column-shear wall seismic elements. Modular light-gauge truss elements on a 5-foot Architect: Marshall & Bowles Engineer: Forell/Elsesser	Roof Plan 
	**Crocker Building San Francisco 1982** An early use of a perimeter-tube welded moment frame to achieve seismic resistance. No interior seismic resisting system. Architect/Engineer: Skidmore, Owings & Merrill	 Photo: SOM

Photo: Skilling, et al (M-K)

Bank of America Tower
Seattle 1982

76-story, 2,000,000 sf in area, 945 ft. tall. Utilized visco-elastic dampers attached to exterior concret-filled super-columns to control drift and to reduce steel weight.

Architect: Chester L. Lindsey
Engineer: Magnusson - Klemencic

Floor Plan

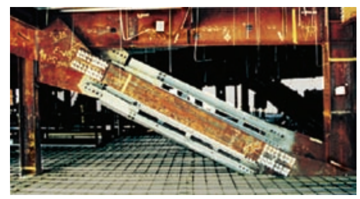


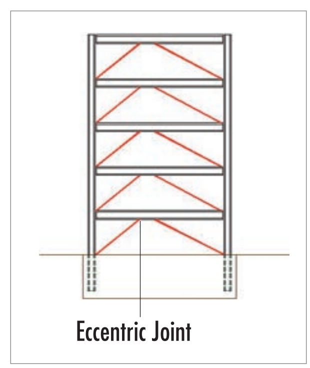
Photo: Skilling, et al. (M-K)

San Jose Federal Building
San Jose,1983

An early use of steel eccentric braced frames with back-up welded moment frames to achieve economy and reliable seismic performance.

Architect: Hellmuth Obata & Kassabaum
Engineer: Forell/Elsesser

This building has experienced 3 moderate to strong earthquakes without damage.



Seismic Structural System **The Creative Years (1985 - 2000)**

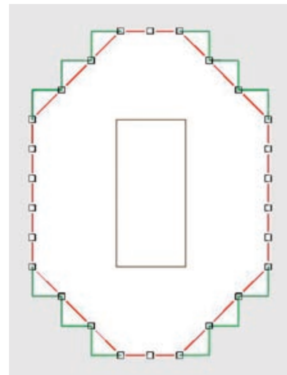
Photo: N. Amin

333 Bush Street San Francisco 1985

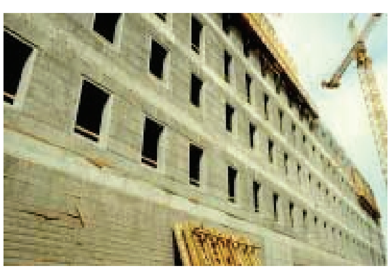
An early version of a dual steel welded perimeter moment frame with a dual interior eccentric-braced frame.

Architect/Engineer:
Skidmore, Owings & Merrill

An excellent combination of systems, combining strength and drift control.



Plan



Life Sciences Building
UC Berkeley 1987

A unique reinforced concrete shear wall building with shear-links between shear walls in both longitudinal and transverse directions, designed to fracture during strong earthquakes. Two interior transverse walls and two exterior perimeter walls designed with diagonal bar cages at the links.

Architect: MBT Architecture
Engineer: Forell/Elsesser

This laboratory building was designed for vibration control and seismic damage control

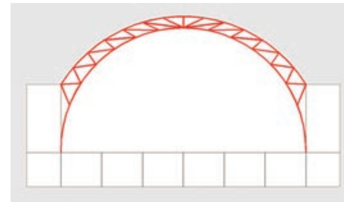
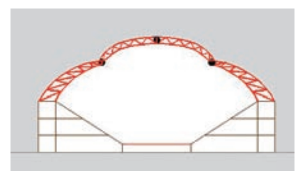
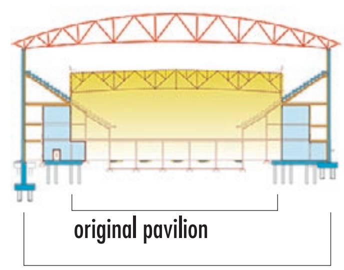
 Photo: Bethleham Pacific	**San Francisco Armory San Francisco 1920** A 150-foot-span trussed arch with perimeter masonry walls. The arch system acts as a semi-rigid and flexible spring diaphragm. Architect: William & John Woole	A conventional solution for long-span roof structures. 
 photo: unknown	**Cow Palace San Francisco 1938** A unique and bold roof system with 3-hinged steel arches, supported on the ends of long steel truss cantilevers located at each side of the arena. The concrete perimeter side and end walls provide primary seismic resistance. Architect: Engineer: Keilberg/Paqutte	The Cow Palace stretched the concept for the roof system by allowing pinned-jointed rotations of the 3-hinged arch to accommodate unplanned differential motions during seismic events. 
 Photo: Robert Canfeld	**Haas Pavilion, UC Berkeley Berkeley 1999** A long-span truss expansion supported on seismic dampers. Architect: Ellerbe Becket Engineer: Forell/Elsesser	
 Photo: SONM	**San Francisco International Airport Terminal 1999** Long-span double cantilever roof trusses with lower floors all supported on seismic isolators. Architect: Skidmore, Owings & Merrill (Greg Hartman) Engineer: Skidmore, Owings & Merrill (Navin Amin)	 Photo: SOM

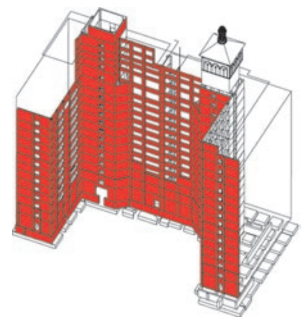
Photo: Robert Canfield

Pacific Gas & Electric
Historic Headquarters San Francisco
1925/1945 1995 Upgrade

The original steel-braced-frame PG&E Building, built in 1925 together with its neighbor, the Matson Building built in 1927, were damaged in the 1989 Loma Prieta earthquake. The strengthening solution joined the two together with a substantial 3-dimensional concrete shear wall and moment frame system designed to be a progressive resistance system, in which 3 elements (moment frames, shear links, and shear walls) yielded sequentially.

Architect: (original) Bakewell & Brown
Engineer: (original) C.H. Snyder
Rehabilitation Engineer: Forell/Elsesser

The new design solution provides basically the essential-facility performance required by PG&E.



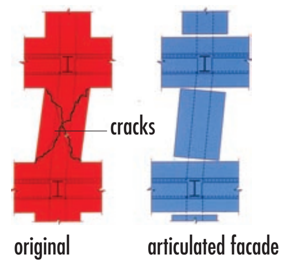


Photo: Robert Canfield

UCSF Biomedical Research
San Francisco 2003

This steel-braced frame utilized unbonded steel braces to achieve ductility with a conventional-braced seismic solution.

The unbonded brace provides equal tension and compression capacity.

Architect: Cesar Pelli & Associates
Engineer: Forell/Elsesser

Photo: Robert Canfield

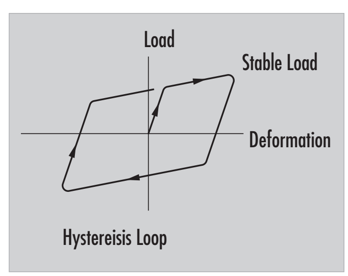


Photo: Robert Canfield

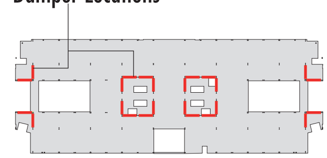
Photo: Robert Canfield

State Office Building
San Francisco 1999

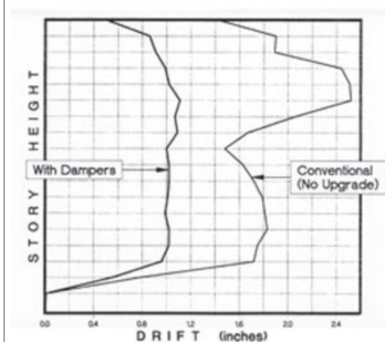
After the 1994 Northridge earthquake, the State of California desired to minimize future damage costs. A dual special steel moment frame acting in conjunction with hydraulic dampers was developed. This system minimizes drift and protects the moment frames.

Architect: Skidmore, Owings & Merrill
Engineer: Forell/Elsesser Engineers

Damper Locations



Floor Plan



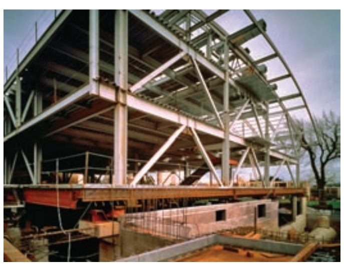
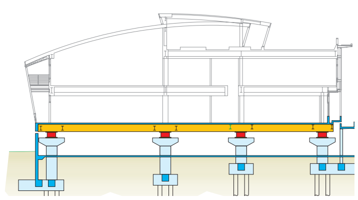


Photo: Robert Canfield



911 Emergency
Communications Center
San Francisco 1998

Seismic performance is critical for this 3-level, 911 Emergency Center. Seismic isolators significantly reduce seismic forces and interstory displacements, allowing a conventional steel braced frame to be used for the superstructure.

Architect: Heller & Manus/
Levy Design Partners Architecture
Engineers: Forell/Elsesser Engineers/)SDI

Photo: Robert Canfield

Seismic isolation for buildings is a relatively new concept. It effectively decouples the building from the shaking ground and significantly reduces the earthquake accelerations and interstory displacements.

7.5 COMMENTARY ON STRUCTURAL FRAMEWORKS

Both the steel-frame and concrete-frame buildings previously tabulated can be summarized on the basis of a primary framing system, considering both frameworks and structural cladding.

7.5.1 Steel Building Frameworks

It can be seen from the previous tabulation that steel frameworks have progressed from a simple steel frame, braced laterally by unreinforced masonry, to complete moment frames with full lateral load resistance. However, the 1994 Northridge earthquake in southern California created serious doubts as to the integrity of welded moment frames. Actually, several years before the 1994 earthquake, thoughtful structural engineers recognized the advantages of dual structural systems for the structural redundancy required to resist large earthquakes. The engineers had two separate tasks. The first task was to minimize the structure for economy, and the second task was to provide a secure load path to protect the structure.

A review of the basic eight steel-frame concepts tabulated in Figure 7-2 indicates that they are all reasonable, but some have substantially more redundancy than others:

1. The simple **1890-1920** steel framework with unreinforced brick infill: The system has vertical support with an infill system that allows brick joint movement for energy dissipation. It is a good inexpensive system that allows for simple repair of brick after an earthquake.

2. The **1910-1930** column-to-girder gusset-plate connection with nominally reinforced concrete infill walls: A good low-cost steel riveted detail with concrete providing stiffness for controlling lateral drift.

3. The **1910-1940** trussed girder wind brace providing inexpensive drift control of the frame: The encasing concrete also provided substantial lateral stiffness, and forced the steel column sections to actually be stronger and stiffer and to create girder yielding, a good contemporary concept.

4. The **1920-1940** knee-braced moment frame with concrete encasement provided a nominally stiff frame system.

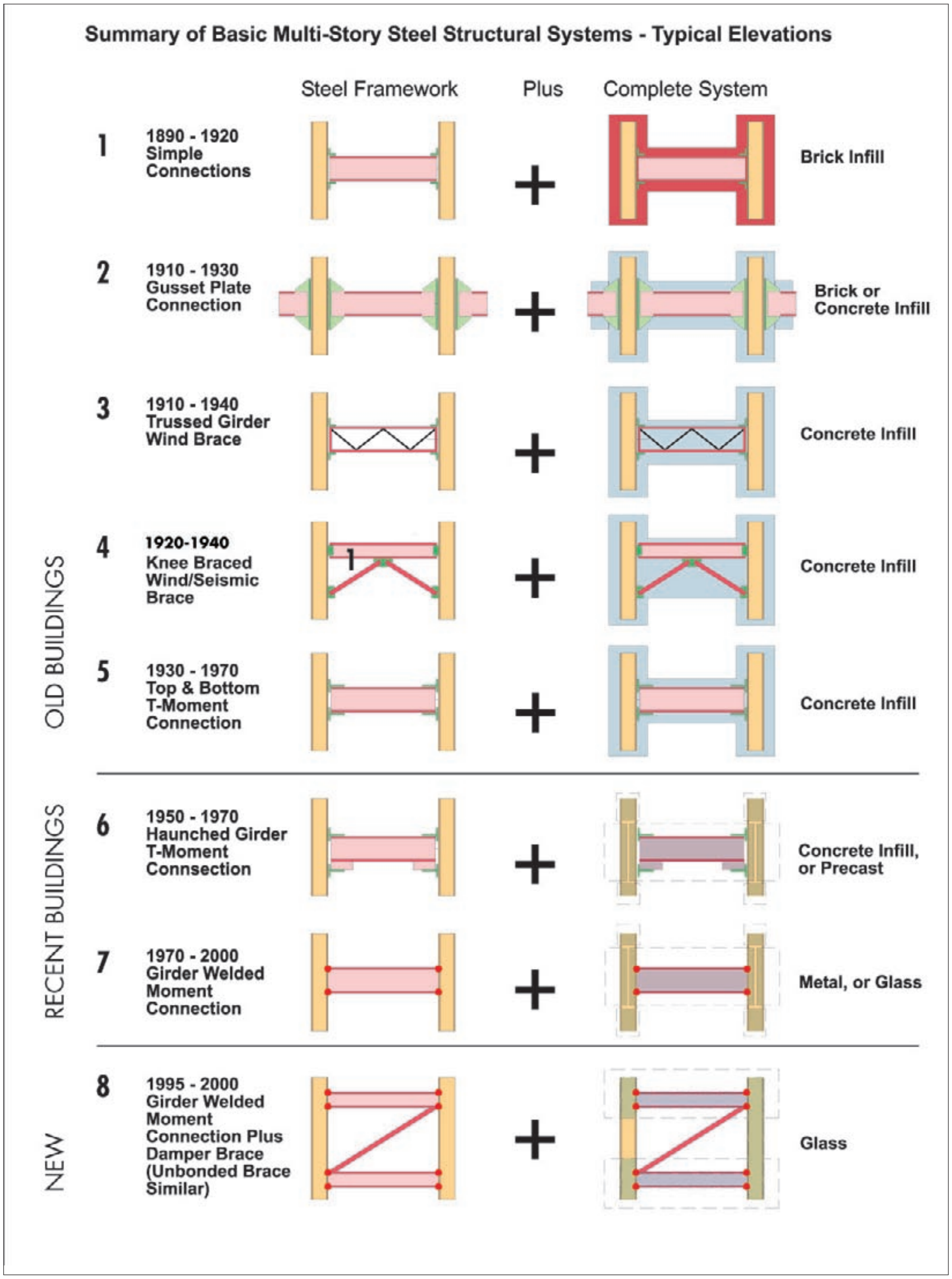


Figure 7-2: Steel building frameworks.

5. The **1930-1970** riveted (or bolted) top and bottom girder fixity to the column created a steel-moment frame. The concrete fire-proofing encasement on buildings through 1960 enhanced the moment-frame stiffness.

6. The **1950-1970** top and bottom bolted haunched girder moment frame provided inexpensive girder stiffness and was especially strong if encased in cast-in-place concrete.

7. The more recent **1970-2000** all welded girder moment frame relied only on the steel system for seismic resistance and produced the most flexible of steel frames. These steel systems were not encased in concrete and were clad only with precast concrete, metal panels, or glass, not providing added structural strength or stiffness.

   After the Northridge earthquake, these conventionally welded frames were seen to be vulnerable, providing far less ductility than anticipated. A major FEMA-funded study has attempted to find solutions to this very significant problem. The current solutions tend to be expensive to fabricate.

8. The **1995-2000** steel-moment frames with a dual system of dampers, unbonded braces, or eccentric-braced frames, all clad with light-weight materials, appear to be good solutions.

The engineering profession had progressed fairly slowly until the early 1980s from the basic framing concepts that were first evolved in the early 1900s. When the concerns about seismic performance and energy dissipation became paramount, researchers and design engineers investigated mechanisms and configurations to supplement the basic rectangular grid framing in use for over 100 years.

7.5.2 Concrete Building Frameworks

Reinforced concrete framing became popular in California immediately after the 1906 earthquake. The significant framing systems that evolved are shown in Figure 7-3.

1. The early years, about **1908 to 1915**, featured nonductile frameworks of reinforced concrete column-girder moment frames. The earliest utilized wood floor infills between concrete floor girders, followed rapidly by all-concrete floors. It was not until the early 1960s that nonductile frames were recognized as an unsafe collapse mechanism, because of inadequate shear capacity, or because of poorly confined concrete.

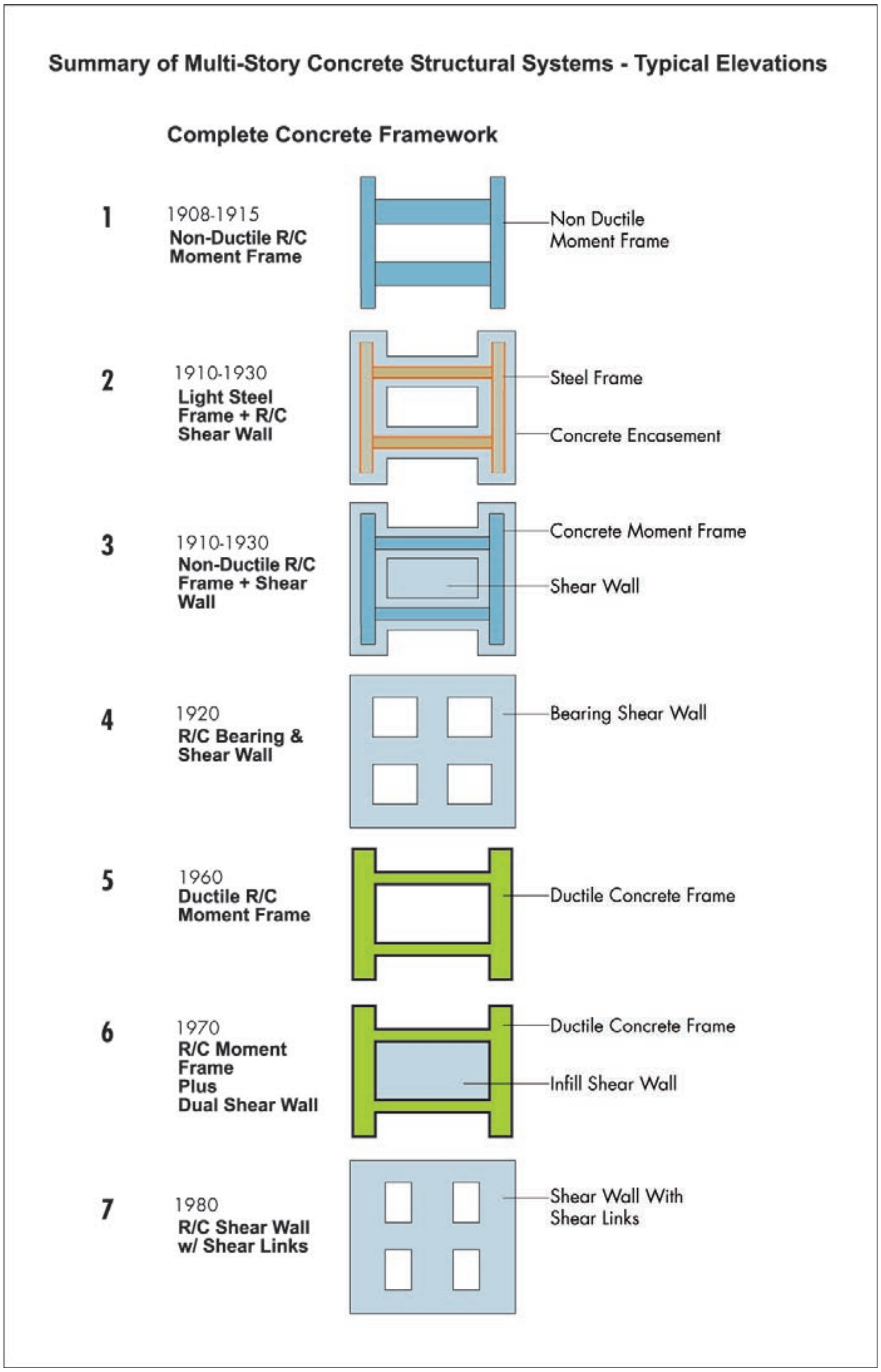


Figure 7-3: Concrete building frameworks

2. Early steel frames for gravity load support were initially encased in masonry cladding, soon to be changed to reinforced concrete frame infill walls, **1910 - 1930**. This was a simple change which added definable strength around the steel.

3. Soon after the bare concrete frames were developed, they were infilled with concrete shear walls for lateral stability, **1910 - 1930**. Not all of these buildings were well conceived structurally, and because of the ease of adding or deleting walls, torsional problems became common for this type of building.

4. Significant reinforced concrete shear-wall buildings acting as bearing walls evolved in the**1920**s, without moment frames.

5. It was not until the late **1960s** and **early 1970s**, after substantial research at the University of Illinois, that the benefit of confined concrete columns and ductile concrete frames was recognized and adopted for dependable seismic resistance.

6. In the **1970s** and **1980s**, a dual system of ductile concrete moment frames coupled with confined concrete shear walls was recognized. This concept works best with a perimeter moment frame and an interior shear wall core, or the reverse—using a perimeter ductile wall and an interior ductile moment frame. Finally, a dependable concrete dual system evolved.

7. Shear walls coupled together with yielding shear links were developed in New Zealand in the **1980**s after successful tests carried out by Park and Paulay in 1975. This coupled-wall system is another excellent example of creative research used to develop a low-cost mechanism for seismic energy dissipation.

7.6 SYSTEM CHARACTERISTICS

Selecting a good structure requires engineering common sense. Common sense requires understanding the earthquake motion and its demands, and understanding the structural behavior of the individual systems available. There are differences of scale (small versus large), differences between elastic and inelastic behavior; and differences of dynamic responses and seismic energy dissipation. Structural and architectural configurations (such as regular versus irregular forms) are also significant in the performance. The many variables often make it difficult to select an appropriate system. The building code lists numerous structural systems, but it does not provide guidance in the selection of a system, and the many systems are not equal in performance.

Key performance issues are elastic behavior, inelastic behavior, and the related cyclic behavior resulting from pushing a structure back and forth. This behavior should be stable, nondegrading, predictable, and capable of dissipating a large amount of seismic energy.

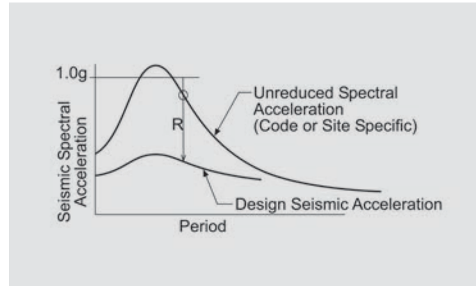


Figure 7-4: Design verses real acceleration.

7.6.1 Elastic Design—Linear Systems

The simple building code approach to seismic design requires diminishing an acceleration spectra plot by use of an R value. Elastic design is expressed by an R value, which is used to modify the acceleration spectral value to a simple seismic design force (Figure 7-4). This is a simple but frequently questionable method. It does not consider performance, nonlinear cyclic behavior, or most important—energy dissipation.

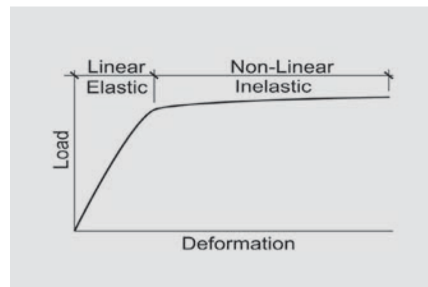
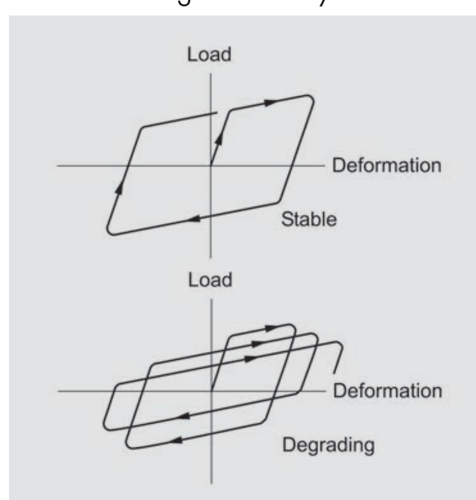


Figure 7-5: Elastic and inelastic.

7.6.2 Post-Elastic Design—Nonlinear Drift

Inelastic design is a better indication of realistic lateral drift or deflection that results from real earthquake motions (Figure 7-5). Nonlinear drift impacts structural and nonstructural behavior. For significant seismic energy dissipation, the drift should be large, but for favorable nonstructural or content behavior this drift should be small. A building with a large but unstable structural drift will collapse. A building with a limited or small structural drift generally will not dissipate significant seismic energy without significant damage.

Figure 7-6: Cyclic behavior.



7.6.3 Cyclic Behavior

A good measure of seismic performance is stable cyclic hysteretic behavior. The plot of load vs. deformation of an element, for motion in both directions, represents cyclic behavior (Figure 7-6). If the load curves are full, undiminished, without “necking down.” they represent a stable system that is ductile and has sufficient capacity to deliver a constant level of energy dissipation during the shaking imposed by an earthquake. Degrading cyclic systems may be acceptable if they degrade slowly and in a predictable manner.

7.6.4 Performance-Based Seismic Design

Years of physical testing and corresponding analytical studies have provided a greatly increased understanding of earthquakes, materials and assemblies. Recent seismic performance design has forced engineers to look at the full range of structural behavior, from linear to nonlinear to failure.

Good, dependable seismic systems behave with a stable, nondegrading behavior; others do not. The ones that are not stable diminish in capacity with continuing cycles, until they have little capacity to dissipate seismic energy. If the earthquake continues for a long duration (over 10 to 15 seconds), the structure can become weak and unstable, and may fail.

The single conventional system with only one means of resisting seismic forces is especially vulnerable to long-duration earthquakes, because when that system degrades, no alternative exists. This concern has evolved for high-performance structures into the concept of utilizing multiple-resisting structural systems that act progressively, so that the overall structural capacity is not significantly diminished during the earthquake.

7.6.5 Nonlinear Performance Comparisons

Considering only strength and displacement (but not energy dissipation), six alternative structural systems are represented by seismic performance "pushover" plots for comparison for a specific four-story building, but a useful example of a common low- to mid-rise building type (Figure 7-7). For other building heights and types, the values will vary. Each system was designed and sized for code compliance, all with a lateral drift of less than one inch. However, the nonlinear displacement (or drift) for each alternative system is significantly different:

- The **braced frame** (BF) needs to reach the 5% damped spectral curve, but the system fails at a drift of about one-and-a-half inches, far below the target of 5%t damping. A possible solution would be to design the brace for about four times the code value; excessively expensive, this solution deals only with strength, not with seismic energy.

- The **shear wall** (SW) needs to reach the 10% damped or spectral curve, but can only reach a drift of about two inches, about half of the required drift. Again, significant over-design might solve the strength problem, but may not solve the energy issue.

- ❍ The **eccentric braced frame** (EBF) needs to reach the 10% damped curve at a drift of about 5.5 inches, which exceeds its ductility and would create excessive deformation. The solution is to add more capacity.

- ❍ The **steel-moment frame** (MF) without a supplemental system needs to reach a 5%t damped curve at a drift of about nine to ten inches, far in excess of its capacity. The only solutions are to over-design the MF or to add a supplemental system.

- ❍ The **steel-moment frame with passive dampers** (MF + Dampers) needs to reach the 20% damped spectra curve which it does at a drift of about four inches. This is a reasonable solution.

- ❍ The **base-isolation** solution (BI) satisfies the demand at a drift from 10 to 12 inches, which it can easily do without damag,e because most of the lateral drift occurs in the isolation itself.

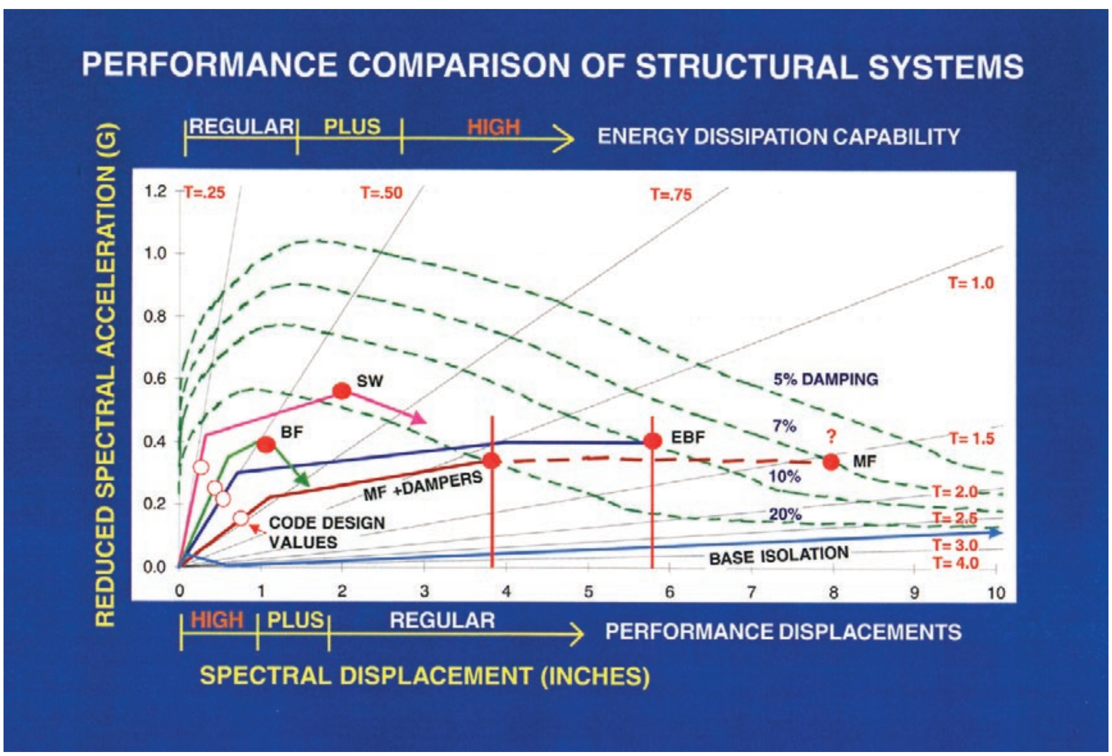


Figure 7-7: Performance characteristics of structural systems.

7.6.6 Energy Dissipation

During the duration of ground shaking, seismic energy is "stored" in a structure. If that input energy can be stored and dissipated without damage to the structure, the behavior is favorable. If, however, the structure cannot store the energy, the system may rupture with either local or catastrophic behavior (Figure 7-8A). Earthquake energy accumulates during ground shaking so that the total energy continually increases. If the structure can safely dissipate energy as rapidly as it is input to the structure, no problem occurs. In fact, the structure can store some of the energy due to ground shaking, and safely dissipate the balance after the earthquake stops. If the structure cannot keep up with the energy input, the structure may suffer minor cracking, or rupture, or major failure (Figure 7-8B).

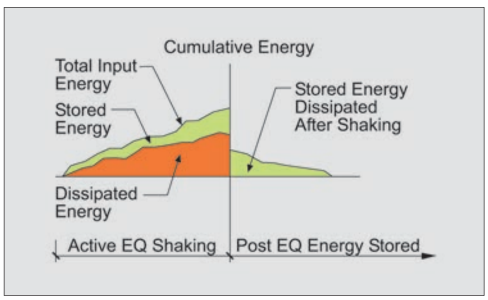


Figure 7-8A: Energy stored.

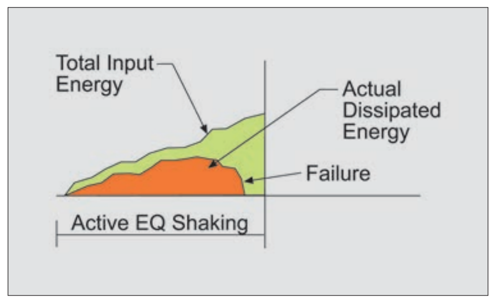


Figure 7-8B: Energy storage failure.

7.6.7 The Design Approach—Force vs. Energy

Searching for the perfect system with conventional solutions has been limited to date, because seismic forces are used that are based on accelerations, and then the resulting lateral drift or movements of the structure are reviewed to check the behavior of both structural and nonstructural elements, such as rigid cladding. This process has an inherent conflict.

Seismic accelerations are used because a design force, **F**, from the relation **F=Ma**, can easily be obtained where **M** is the building mass, and **a** is the ground acceleration.

For the typical stiff-soil site, the larger the acceleration **a**, the larger the seismic force **F**. The larger the force **F**, the stronger the structure is; the stronger the structure, the stiffer the structure. The stiffer the structure is, the higher the seismic acceleration, and so on. A strong, stiff structure appears to be good conceptually. The lateral drift (or horizontal displacement) is at a minimum, which is also good. However, the contradiction of this approach comes from seismic energy dissipation, which is the fundamental need and characteristic of good seismic design; energy dissipation comes from large displacements, not small displacements.

Large displacements are needed to dissipate seismic energy, and **small displacements** are needed for lateral drift control to protect cladding, glazing, and interior systems. This produces a conflict for most of the classic conventional structural concepts and normal nonstructural components.

The most useful **seismic systems** are those that have predictable, stable, nondegrading cyclic behavior. Contemporary structures with these characteristics are base-isolation systems; moment frames with dampers; shear-link systems, such as coupled shear walls and eccentric-braced frames; and other dual-resistance systems with built-in redundancy.

The most useful **nonstructural components** are those that accommodate large lateral movements without failure.

7.7 THE SEARCH FOR THE PERFECT SEISMIC SYSTEM

The 100-year review of seismic systems indicates a slow development of structural solutions. In fact, most new development occurs after a damaging earthquake. Perhaps this slow periodic development is due to the need to discern whether previous ideas were successes or failures. The following topics represent steps and ideas in the search for the "perfect system."

7.7.1 Structural Mechanisms

There have been periodic significant conceptual breakthroughs in structural thinking. For example, Dr. Ian Skinner and his team, working at the New Zealand National Laboratory, developed in 1976 a set of energy dissipating concepts suitable for use in seismic protection. Among the most notable concepts was the practical use of elastomeric isolation bearings for global protection of complete structures. Some of the other concepts

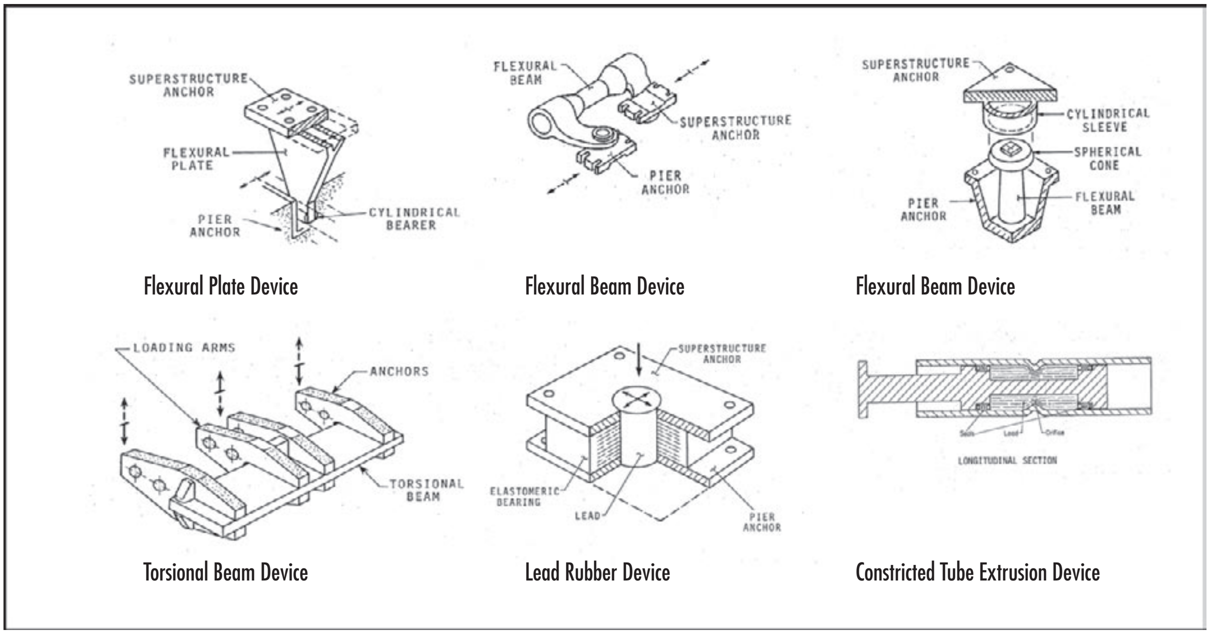


Figure 7-9: A set of energy dissipating devices.

SOURCE: NEW ZEALAND NATIONAL LABORATORIES

developed in New Zealand have also been utilized in building design. Figure 7-9 illustrates the set of mechanisms published in 1980.

Many of these relatively simple mechanisms were new to the design professions and to constructors; consequently, they represented unknown processes and unknown costs. Development and education are the keys to the acceptance and adoption of these systems. Perhaps a few well-publicized, prototype projects would familiarize the design professions and constructors with the details and costs of these good ideas. To date, this has only been done with the flexural plate and the lead-rubber isolation bearing.

7.7.2 Semi-Active and Active Dampers

Recent research and design work in Japan and the United States has focused on approaches for structural control during actual wind storms and earthquakes. These approaches have been summarized by Hanson and Soong, and can be divided into three groups: **passive systems**, such as base isolation and supplementary energy dissipation devices; **active**

systems that require the active participation of mechanical devices whose characteristics are used to change the building response; and **hybrid systems** that combine passive and active systems in a manner such that the safety of the building is not compromised, even if the active system fails. The current systems being studied are characterized by devices that control properties and behaviors. Some of these systems are in limited use; others are still in development.

The goal of these devices is to respond actively to the variable character of wind and earthquake displacements, velocity, accelerations, etc., by adding damping or altering stiffness. This controlled behavior will provide the needed resistance to respond to ever-changing earthquakes. However, the challenge with semi-active or active control systems is maintaining their behavior trouble-free over an extended period of time - specifically over many years. These concepts, although very appealing, will require some time to perfect and bring to market.

7.7.3 Cost-Effective Systems

The most important measure of good earthquake-resistant design is the impact on the structure after the earth has stopped shaking. With little building damage, repair costs are low. With significant damage or collapse, repair or replacement costs are high. The measure of success in seismic design is selection of a structure that will suffer minimum damage, and with corresponding low post-earthquake repair cost. The behavior of each structural system differs with earthquake ground motions, soil types, duration of strong shaking, etc. Our past observations of damage yields the best measure of future repair costs. Systems with stable cyclic behavior, good energy dissipation, and controlled interstory drift will yield low repair costs (Figure 7-10).

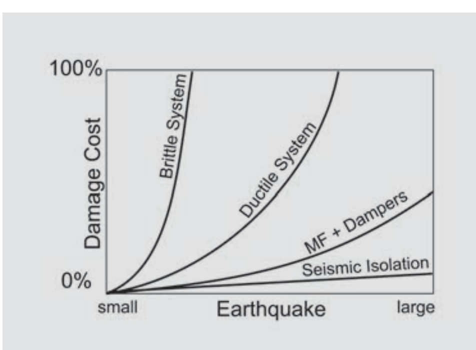


Figure 7-10
System performance.

Seismic - Structural System Characteristics

System	Seismic Design R	Non-Linear Drift	Cyclic Behavior	Energy Dissapation	Post EQ Repair Cost
UR Masonry Walls	1.5	Medium to Collapse	Unstable	Low	High
Timber Framing	5.5	Medium	Stable	Medium to High	Medium
Steel Frame URM Wall	(3.5)	Medium	Stable	Medium	Medium
Steel Frame + R/C Walls	5.5	Medium	Stable	Medium	Medium
Non-Ductile R/C - MF	3.5	Large to Collapse	Unstable	Low	High
Steel Frame + Braces	4.5	Large to Collapse	Unstable	Low	High
R/C Shear Walls	4.5	Medium	Stable to Unstable	Medium to High	Medium to High
RG Masonry Walls	4.5	Medium to Large	Stable to Unstable	Medium	Medium to High
Non-Ductile Steel MF	4.5	Medium to Large	Unstable	Medium	High
Composite Timber & Steel	4.4	Medium	Stable	Medium to High	Low

Figure 7-11A: Structural seismic characteristics.

Seismic - Structural System Characteristics

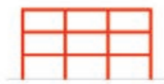
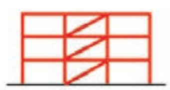
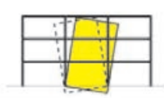
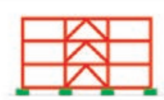
System		Seismic Design R	Non-Linear Drift	Cyclic Behavior	Energy Dissapation	Post EQ Repair Cost
RG Masonry R/C MF		**5.5**	**Medium**	**Stable to Semi-Stable**	**Medium to High**	**Low to Medium**
Ductile Steel MF		**8.5**	**Medium to Large**	**Stable to Semi-Stable**	**Medium**	**Medium to High**
Ductile R/C MF		**8.5**	**Medium to large**	**Stable to Semi-Stable**	**Medium**	**Medium to High**
Steel Eccentric Brace		**7.0**	**Low to Medium**	**Stable**	**Medium to High**	**Low to Medium**
R/C Link Shearwall		**6**	**Medium**	**Stable**	**Medium to High**	**Low to Medium**
Dampers + Steel MF		**8.5**	**Low to Medium**	**Stable**	**Very High to High**	**low**
Unbonded Steel Brace		**6.5**	**Medium**	**Stable**	**Medium to High**	**Low to Medium**
Rocking System		**(8.5)**	**Large Rocking Motion**	**Stable**	**High**	**Low**
3-Part Progressive System		**8.5**	**Low to Medium**	**Stable**	**High**	**Low**
Seismic Isolation		**(1.6-2.0)**	**Low Interstory Drift**	**Stable**	**Very High**	**Very Low**

Figure 7-11B: Structural seismic characteristics.

The above five attributes, which are tabulated in Figures 7-11A and 7-11B for the various significant seismic structural systems, show that a range of possibilities is indicated. These tables are presented for discussion, and the evaluations stated are the author's opinion, based on observations, analysis, and common goals for seismic performance. The green colored boxes are favorable conditions; the red tend to be unfavorable. The favorable structural systems will do the following:

- Possess stable cyclic behavior
- Control lateral drift
- Dissipate seismic energy without failure
- Create a low post-earthquake repair cost

The design reduction value R, discussed above in section 7.6.1, does not necessarily correlate with performance. The "R" value was a consensus value developed for conventional elastic design. With the advent of performance design based on nonlinear evaluation the R value serves only as a "rough estimate" of system behavior, but not a realistic estimate of performance.

7.7.4 Avoiding the Same Mistakes

Architects and engineers learn from their detailed investigations of past earthquake damage and can document the significant issues and lessons that can be learned about particular structural problems. Some problems occur because of inappropriate building or structural configuration, some because of brittle, nonductile structural systems, some because the building or structure could not dissipate sufficient seismic energy, and some because of excessive loads caused by dynamic resonance between the ground shaking and the building. Figure 7-12 illustrates some classic problems.

Why, with all our accumulated knowledge, does all this failure continue? Buildings tend to be constructed essentially in the same manner, even after an earthquake. It takes a significant effort to change habits, styles, techniques and construction. Sometimes bad seismic ideas get passed on without too much investigation and modification.

Figure 7-12: Examples of typical earthquake damage.

The challenge for architects, engineers, and constructors is to ask why so much damage occurred, and what can be done to correct the problem. Understanding the basic seismic energy demand is a critical first step in the search for significant improvement. Not all structural systems, even though they are conventional and commonplace, will provide safe and economical solutions.

7.7.5 Configurations Are Critical

Configuration, or the three-dimensional form of a building, frequently is the governing factor in the ultimate seismic behavior of a particular structure. Chapter 5 covers conventional configuration issues where conventional rectangular grids are used in building layout, design and construction. However, contemporary architectural design is changing, in large part because the computer allows complex graphic forms and analyses to be generated and easily integrated into a building design.

The resulting irregular, random, free-form grids and systems have just begun to be explored from the structural engineering viewpoint. They are frequently rejected because of various cost issues, and because of unproven real earthquake behavior.

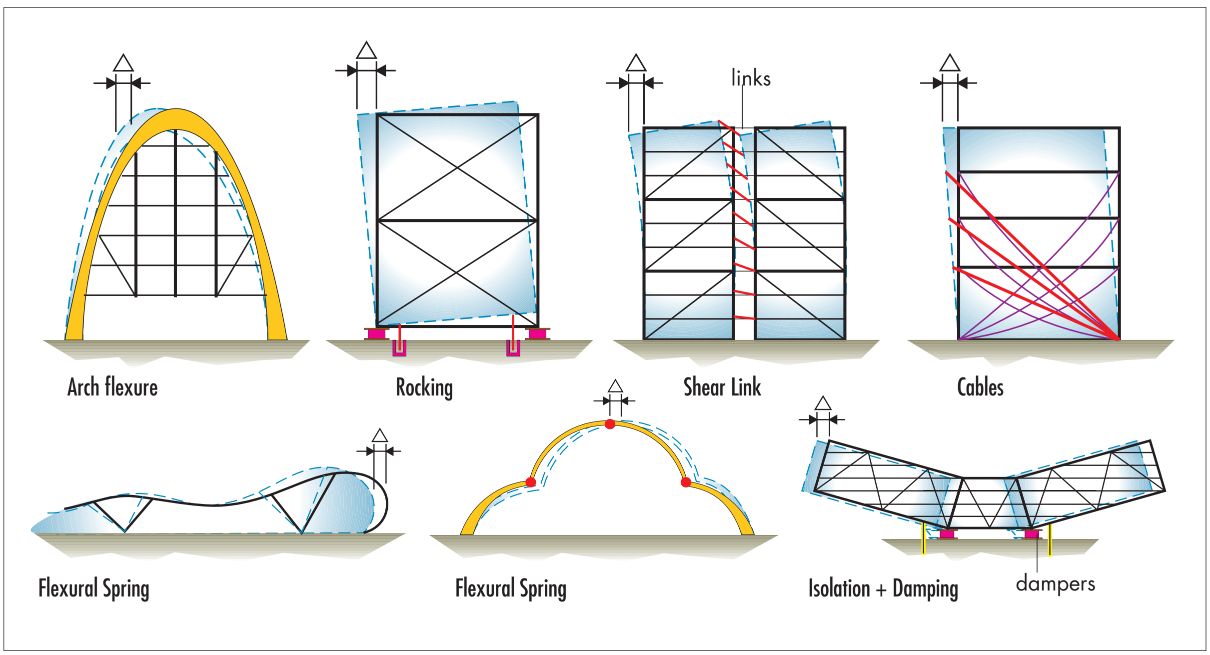
The potential for optimizing seismic resistance with respect to structural configuration is an obvious direction for the future. Structural form should follow the needs. How can we define seismic needs?

Buildings must dissipate energy; how does one configure a structure to dissipate energy? There are natural forms and design concepts that act as springs, rocking mechanisms, flexural stories, yielding links, articulated cable-restrained configurations, pyramid forms, cable anchors, etc. Any system that can dissipate seismic energy without damage is a candidate. Figure 7-13 illustrates some special concepts utilizing building configuration to dissipate energy.

7.7.6 Common-Sense Structural Design-Lessons Learned

The simple way to reduce seismic demand within a structure is to understand the actual demand. An earthquake is a dynamic phenomenon with all its classic characteristics. If one can reduce the effective damaging character of the earthquake, the behavior of a structure or building will be significantly improved. The following five issues can significantly reduce earthquake damage and related costs.

Figure 7-13: Concepts that use building configurations to dissipate energy.



● Select the Appropriate Scale

The size and scale of the building should determine the appropriate structural solution. It is common sense to use a light braced frame for a small structure, such as one- or two-story wood or light-gauge steel systems. The light seismic system is compatible with the light building mass. As the building mass increases with the use of heavy concrete floors, the mass increases by a factor of 8 to 12 times. This would imply a much heavier seismic frame or brace (Figure 7-14), 8 to 12 times stronger per floor, and if the building grows from two to ten stories, the total mass increases from two to 80-120 times.

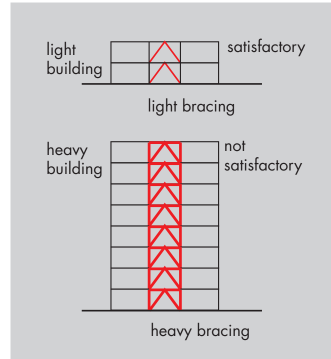


Figure 7-14: Building scale.

The same frame or brace which works well at two stories no longer works well at ten stories, because structural components when simply scaled up no longer behave in the same fashion. For instance, a light steel section with 3/8-inch flanges is ductile, but no longer has the same ductility when the flanges are two inches thick. The thick sections and welds are now subjected to high shears and are prone to failure.

Some structural systems are very forgiving at small sizes but not at a large scale. Alternative structural systems must be used for the larger, more demanding buildings. The appropriate systems must not degrade in strength and should have ample sustainable damping. It is critical that scale be considered, even though it is not considered by the seismic building codes. This scale issue requires study, observation, and common sense, but the issue of appropriate scale continually confronts designers. Careful, unbiased research is necessary.

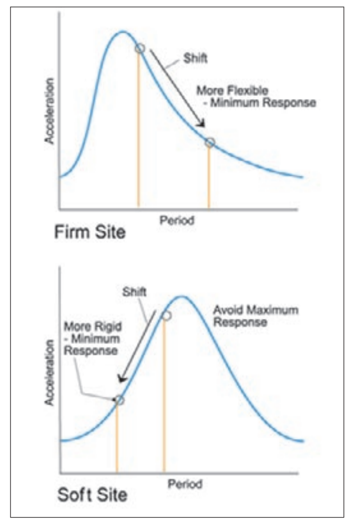
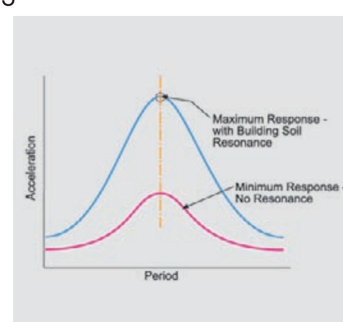


Figure 7-15: Building resonance.

● Reduce Dynamic Resonance

It is important to significantly reduce the dynamic resonance between the shaking ground and the shaking building, and to design the structure to have a period of vibration that is different from that of the ground period. The difference is simply illustrated with a classic resonance curve (Figure 7-15).

Figure 7-16: Effect of resonance.



The relationship between the building period and spectral acceleration of a real earthquake varies between a site with firm soil and a site with soft soil (Figure 7-16).

If the building has a period of vibration, T1, corresponding to the peak ground acceleration, the most severe demand occurs. Shift the period, T2, by altering the structural system to lengthen the period (firm soil) or shorten or lengthen the period T2 (soft soil).

This requires design effort by both the architect and engineer, but the result can be a significant reduction in the seismic response.

Increase Damping

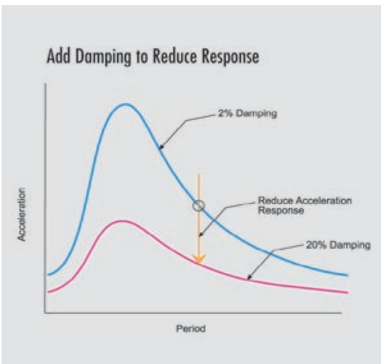


Figure 7-17: Reduction of demand by damping.

It is valuable to significantly increase the structural system damping. Damping reduces vibration amplitude similar to the hydraulic shock absorbers in an automobile, and damping reduces the structural demand (Figure 7-17), as illustrated in the spectral acceleration plot.

Significant damping can be introduced into a structure by 1) adding a passive damping system (hydraulic, friction, etc.); 2) utilizing a fractured concrete member such as a shear-link serving to couple two walls; 3) fracturing a concrete shear wall; 4) utilizing a seismic isolation system (with 10 to 20% damping), or 5) utilizing a tuned-mass damper (a challenging solution for most buildings). An increase of damping by using a non-distinctive system is a most positive solution for reducing seismic demand.

Provide Redundancy

It is also important to add redundancy or multiple load paths to the structure to improve seismic resistance. After experience in many earthquakes and much study and discussion, the engineering profession has generally concluded that more than a single system is the ideal solution for successful seismic resistance. If carefully selected, multiple systems can each serve a purpose; one to add damping and to limit deflection or drift, the other to provide strength. Multiple systems also serve to protect the entire structure by allowing failure of some elements without endangering the total building.

An informative sketch of a classic redundant-framing concept, with frames on each grid line, versus a contemporary multiple system with two types of framing, one for strength, the other for damping, is shown in Figure 7-18. The current dual systems now being developed and utilized are a significant improvement over the historic single seismic resisting systems.

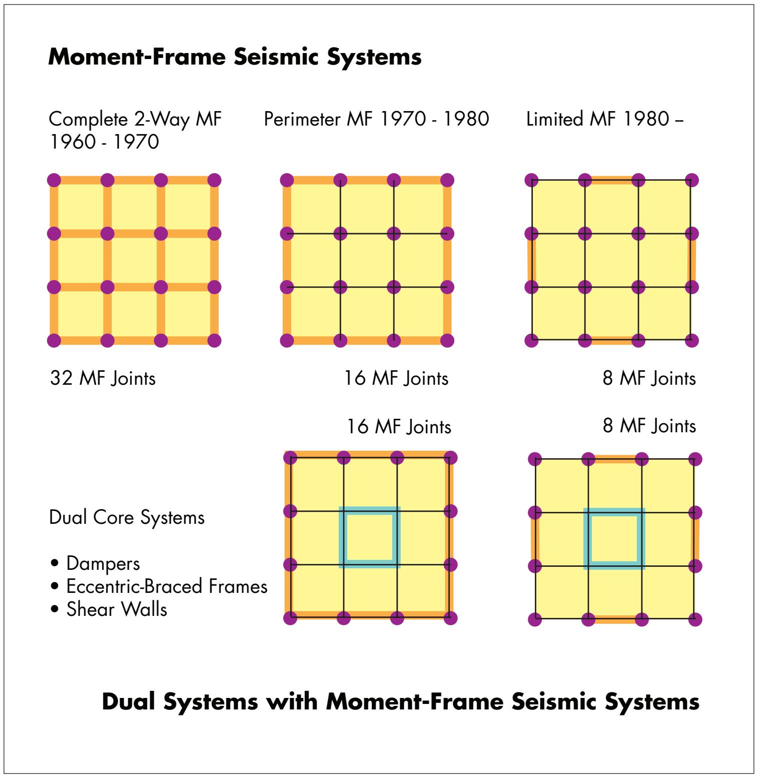


Figure 7-18: Single and multiple system concepts.

Energy Dissipation

Solving the seismic energy dissipation problem is the ultimate test of good earthquake-resistant design. Since large building displacement is required for good energy dissipation, while minimum displacement is required to protect the many brittle non-structural components in the building, only one seismic resisting system adequately solves both aspects of this problem—seismic base isolation.

However, seismic isolation is not suited for all building conditions: specifically, tight urban sites adjacent to property lines (a movement problem), and tall buildings over 12 stories (a dynamic resonance problem). Cost of construction is another consideration (about a 1 to 2% premium). Seismic isolation is an ideal solution for irregular buildings and unusual or creative building forms that are difficult to solve with conventional structural systems.

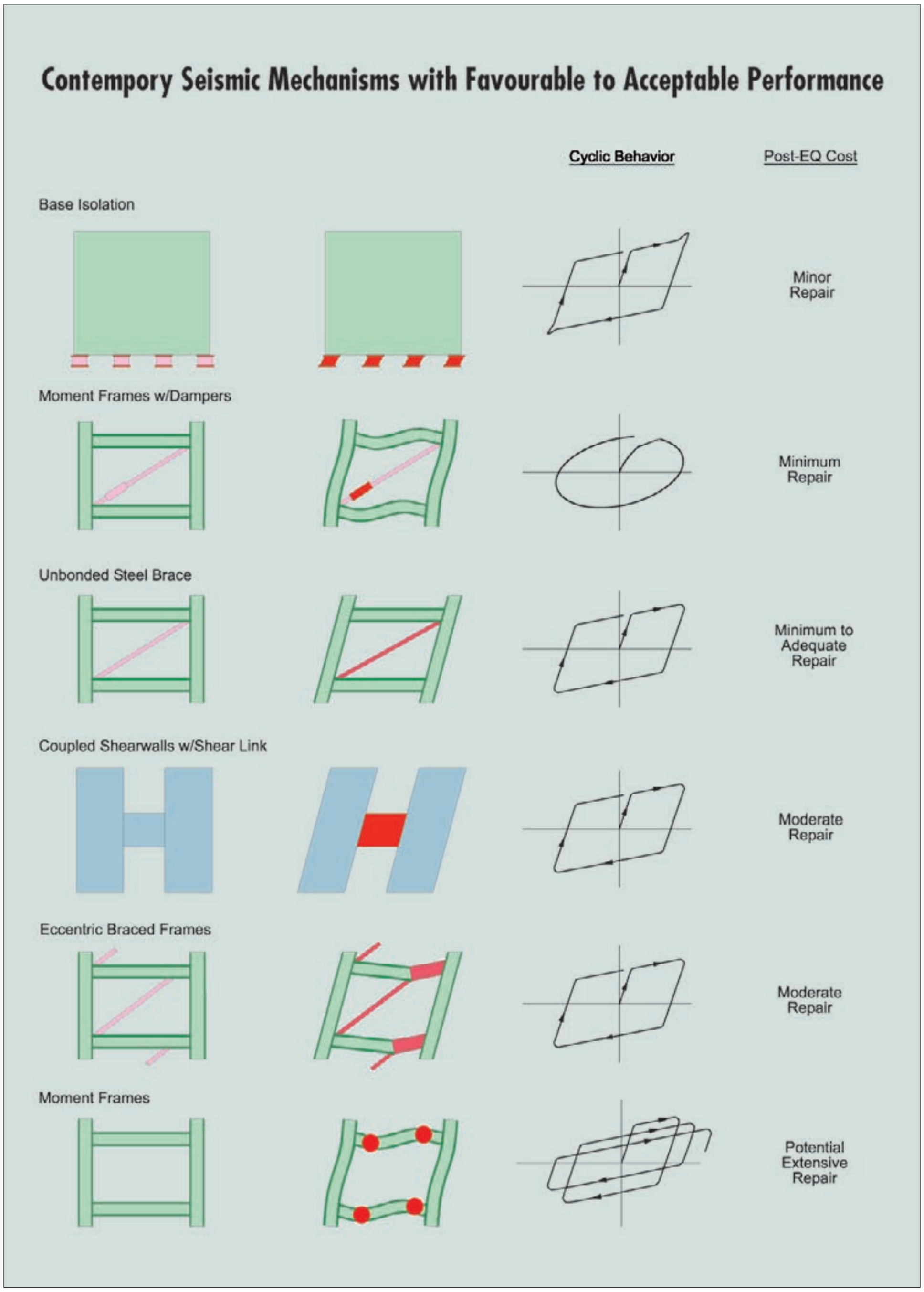


Figure 7-19: Six high-performance seismic structures.

Other dual systems can also solve the energy dissipation problem. Six seismic systems are outlined in Figure 7-19. Each system has a cyclic load-deformation loop which is full and nondegrading. Most importantly, each has a predicted minor-to-moderate post-earthquake damage, with a correspondingly low post-earthquake repair cost. This fact makes these good seismically sustainable buildings, expanding the meaning of "green building."

7.8 CONCLUSIONS

We must continue to develop and use increasingly realistic analysis methods to design new buildings and to modify existing structures. This is not a simple task because of the numerous variables: duration of shaking, frequency content of the seismic motion, displacement, velocity, acceleration, direction of the earthquake pulse, proximity to the active fault, and soil amplification effects. In addition to the ground effects, the structure has its own variables: size; shape; mass; period of vibration; irregularities in shape, stiffness, and strength; and variation in damping.

After the past 100 years of seismic design, the perfect architectural/structural solution is still elusive, but with creative thought, testing, and computers, the problem is more transparent, and we can now more rapidly study variables than in the recent past. Design work today has become part research, part invention, and common sense, and we are on the threshold of creative breakthrough.

7.9 REFERENCES

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