FEDERAL EMERGENCY MANAGEMENT AGENCY

INFRASTRUCTURE SUPPORT DIVISION

FEMA TASK FORCE REPORT

SAN FRANCISCO CITY HALL

THIRD APPEAL

May 27, 1994

Subgrantee: City and County of San Francisco
Disaster # 845
Loma Prieta Earthquake

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FEMA RESPONSE AND RECOVERY DIRECTORATE TASK FORCE REPORT

SAN FRANCISCO CITY HALL

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CITY AND COUNTY OF SAN FRANCISCO CITY HALL

Third Appeal
FEMA-845-DR-075-00000

SECTION I:

INTRODUCTORY BACKGROUND

A. Building Description

The San Francisco City Hall, a familiar San Francisco landmark, is a block-sized monumental four story structure with a large central dome. The building is constructed with a steel frame, with masonry infill. The exterior facade is granite, and the major central rotunda on the interior is finished in limestone, with the upper parts of the walls and the interior surface of the dome finished with ornamental plaster. The masonry infill consists of brick, forming the exterior facade and courtyard walls (as a backing to the granite on the facades), and some of the major interior walls. Most of the interior partitions are constructed of four inch hollow clay tile, with a plaster finish on both sides.

The City Hall forms a figure "8" with two interior light courts located on either side of the central dome and rotunda. The city offices occupy the first three floors, and municipal courtrooms and law library completely fill the fourth. The ornate city council chamber is located on the second floor, and large office areas open to the public for court and city tax records are located on the first floor.

The San Francisco City Hall is an historic structure listed on the National Register of Historic Places, and thus any project funded in whole or in part with Federal funds is subject to review under section 106 of the National Historic Preservation Act.

B. Historical Background

The San Francisco City Hall was designed and constructed immediately following the great San Francisco Earthquake and Fire of 1906. It was intended to be a symbol of the re-emergence of the city, a large part of which had been leveled by that catastrophe. The City

Hall's predecessor, which was also a monumental building with a tall dome, was constructed of unreinforced masonry. Only the dome itself was supported by a steel frame. The 1906 earthquake and fire left this earlier city hall building completely in ruins. Only the steel frame structure of the dome remained standing, with the building itself in a state of collapse around it.

The present City Hall was designed immediately following the dramatic collapse of this earlier large and monumental city hall. Doubtlessly at the time, the architects and engineers of this new building were particularly conscious of the potentially devastating effects of earthquakes, and the designs for the building were the state of the art at that time. The new City Hall was constructed with a complete steel frame, with the masonry infilling the frame, rather than with masonry alone. Contemporary engineering publications describe the ideas which the engineer, Christopher H. Snyder, carried out in his design to deal with the earthquake threat, including his use of a "flexible story" concept at the first floor level.

C. The earthquake damage

The epicenter for the October 17, 1989 Loma Prieta Earthquake, was located approximately 60 miles to the south of San Francisco. This earthquake had varying affects on San Francisco depending primarily on the nature of the underlying soil conditions. Because of the distance of the epicenter of the earthquake from the city, soft soil areas that had greater surface response because of their tendency to resonate with the longer seismic waves. Pockets of significant damage in the city were located on fill or alluvium. Less significant damage extended through other areas located on firmer ground. The City Hall is located on an area of "Quaternary dune sand," a "fine grey sand underlain by a tough blue clay stratum." The site lies inland from the areas of San Francisco located on fill over bay mud where greater damage occurred. The City Hall site is firmer than the bay mud locations, with "no historical evidence of soil liquefaction" for the site. (Forell/Elsesser (F/E) Report, p4-4).

The damage to the building consisted of the following:

1) Cracks and slight dislocation of some of the exterior masonry. The cracks are located for the most part along the mortar joints of the exterior granite, with evidence of cracking through brick masonry units on the inside face of the walls.

2) A crack in one granite lintel spanning between exterior columns on the north side of the building.

3) Hairline cracks in many of the interior hollow clay tile (HCT) walls, visible through the plaster surface, most frequently along the mortar joints between the

units and at the column to HCT infill interface.

4) Displacement of hollow clay tile units in a few of the hollow clay tile walls at the first floor level in areas where the HCT is not bounded and restrained by the building frame.

5) Displacement of the marble cladding at the sides of the staircase and hallways in the central rotunda area. The staircase walls have been temporarily supported and covered with plywood making their inspection by the Task Force incomplete.

6) Slight movement and displacement of some of the limestone cladding on the piers and arches of the central rotunda. Some of the keystones in the first story arches have dropped slightly.

7) Minor cracking in the ornamental plaster ceiling of the central rotunda, particularly at the intersections of the walls and ceiling. Some small pieces of ornamental plaster broke off, falling to the floor (there were no reports of injuries).

8) Widening of existing cracks, with some additional new cracks, in the floor slabs, particularly around the central rotunda and around the dome.

9) Some broken glass in the windows of the central rotunda.

There was no reported damage to any part of the steel frame. All of the damage was limited to the cracking of brittle materials such as masonry and concrete. There were no reports of major falling elements, although some smaller pieces of plaster fell from the cracks in the ceilings and walls. The building was determined to be structurally safe by city inspectors, with a sign installed after the earthquake stating that it was safe. The building was not vacated at any time after the earthquake, and has been occupied continuously since the earthquake.

D. FEMA ACTION:

1. Damage Survey Reports:

The first category E (permanent repairs) Damage Survey Report (DSR) is dated April 16, 1990. This DSR approves $2.6 million for miscellaneous interior repairs. This was followed by two other DSRs for permanent repairs, one for $1 million, and another for a little under $124,000. No other DSRs for the repairs have been approved because of the City's ongoing appeals to have the seismic upgrade of the building made eligible.

FEMA has also approved funds for the assessment of the damage and the preparation of the engineering report on the building by Forell/Elsesser Engineers (F/E). In a letter dated December 15, 1992, the City of San Francisco agreed to limit charges to FEMA for this work to $200,000 until the San Francisco Building Code (SFBC) section 104(f) issue has been resolved. The F/E report, Earthquake Damage and Repair Study of San Francisco City Hall, was completed in August, 1991.

2. First Appeal: July-September, 1992

On July 30, 1992, the State forwarded the city's first appeal asking for the full costs of the seismic upgrade of the City Hall. The primary argument in the city's appeal letter was that the addition of the words "or lateral" was an "interpretation" of the SFBC which was within the building official's authority. They argued that this "interpretation" was in agreement with the intent of the original code and also was the only rational reading of the section of the code in question. The FEMA, Region IX response to this point acknowledged the authority of the superintendent to make "interpretations" of the code, but concluded that, in this case, the added words changed the meaning and intent of the code section to such an extent as to exceed the authority of the building official to make changes without city council approval.

The City argued that their proposal to do a full seismic upgrade was in compliance with the intent of Congress when it included restoration "...in conformity with current applicable codes, specifications, and standards (including hazard mitigation criteria required by the President...)" in the definition of net eligible costs. The FEMA answer was that the current applicable codes did not require the upgrade and that the existence of a code for new construction did not mean that it applied to situations involving the repair to existing buildings of an earlier technology.

As further justification for their request, the city stated that the Regional Director should exercise his discretionary authority under the hazard mitigation section of the regulations by providing funding for the full upgrade. In response, FEMA pointed out that the regulations require that the hazard mitigation measures be cost-effective (benefits exceed costs) and that the City had not submitted any justification that the proposed base isolation project would be cost-effective. Therefore, FEMA had no basis on which to approve the proposed seismic upgrade design as a hazard mitigation measure.

For the above reasons, the Regional Director of Region IX denied the appeal. This denial was sent by letter to the State Office of Emergency Services (OES) on

September 25, 1992.

Damage Survey Reports for a portion of the repairs of the earthquake damage were approved in 1990 for the total amount of approximately $3.8 million. Subsequent repairs have been delayed pending the plans for the structural upgrading of the building. Currently the City of San Francisco is completing the design and working drawings for the structural upgrade of the building, including base-isolation.

3. Second Appeal, filed January 6, 1993

The second appeal was submitted to FEMA by the State on January 6, 1993. Enclosed with the State letter was a brief from the City of San Francisco Chief Administrative Officer, Rudolf Nothenberg, which detailed the City's position on each of the issues. The issues and arguments in this second appeal were basically the same as in the first appeal with some additional issues. The City's claims are the following, and the FEMA response to each claim is contained in Section II of this report.

1) that the building official's interpretation adding "*or lateral*" to the language of the following section of the SFBC has the force of law, and that the work needed to repair the earthquake damage would trigger this altered provision of the code. (This is the original basis for the claim)

   > *SFBC Section 104(b)B(ii) [a building must be seismically upgraded] "when more than 30% of the floor and roof areas of the building...will be involved in substantial structural.... repair work.... The areas to be counted towards the 30% shall be those areas tributary to the vertical* ***(or lateral)*** *load carrying components that...will be removed, added or altered.*

2) that the repairs would trigger an upgrade under the unaltered language of the SFBC based on the position that the repair of damage to areas "tributary to the vertical load carrying structure" would meet the code's criteria, and other provisions of the SFBC.

3) that the <u>State Historical Building Code</u> (SHBC) itself requires an upgrade, independent of the language of the SFBC.

4) that, because FEMA had funded the cost of producing the engineering study of the San Francisco City Hall carried out by Forell/Elsesser Engineers of San Francisco, FEMA should follow its findings and recommendations.

5) that the upgrade of the San Francisco City Hall will be "cost-effective in accordance with the Stafford Act and 44 C.F.R. 206.226(c)" because of stated expectations that the un-upgraded building would be destroyed in a "great" earthquake, with total costs which would "easily exceed $600 million."

As in the first appeal, the city also contended that the upgrade should be eligible as a hazard mitigation measure in accordance with § 206.226(c)-Hazard Mitigation. They claim that such a measure would be cost effective in view of the high occupancy and civic importance of the City Hall. Later in the letter, brief mention is made of benefits versus costs. It is claimed that the building would be subject to major damage in the event of a moderate earthquake or collapse in the event of a "great" earthquake. In their letter, it was postulated that a "moderate" event could cause substantial damage, whose repair cost would equal the cost of the upgrade in the opinion of the F/E engineers. That cost was estimated at $85 to $105 million. It should be noted that the Loma Prieta Earthquake, which had an estimated local ground acceleration of about .10g, caused about $15 million in damage.

F/E also estimated that in the event of collapse, the replacement cost of the entire building would be about $430-$500 million, approximately 8-10 times the cost of a premium new office building of similar square footage and function, or 2-3 times the cost of a new monumental city hall style building.

To further examine the issues raised in the two appeals, FEMA engaged two consulting teams. The first, Preece, Goudie and Associates, Engineers (PGA) were engaged to review the findings of the Forell/Elsesser Report, particularly in regards to the code "triggers." The second was VSP Associates, Inc., (later two principles formed a separate firm called Goettle and Horner, Associates (G&E)) to provide a benefit/cost analysis of the various upgrade options.

A report on the findings of the Second Appeal was completed by FEMA, but never issued pending the re-review of the appeal by a Task Force appointed by the Director. The Second Appeal Report included the finding that a full building upgrade of the San Francisco City Hall was not triggered by either the SFBC or the SHBC, or any other applicable code or ordinance, and the recommendation that FEMA provide discretionary funding for hazard mitigation based on a more more modest life-safety upgrade proposal. The Report also concluded that the benefits of the City's proposed base isolation design fell far short of its approximately $180 million in costs. The Report of the consulting Engineers to FEMA, Preece Goudie and Associates, and a letter to the State of California by John Kariotis, Engineer, which formed a part of the Second Appeal Report, are included in the Appendix of this report.

4. FEMA S.F. City Hall Task Force - Third Appeal

Prior to issuance of the Second Appeal findings, Director James Lee Witt appointed a Task Forcewith the concurrance of the grantee and subgrantee to provide an independent review of the City's appeal at the third level. The principal charge of this task force was to review 1) the expected future earthquake damage and life safety risks associated with the SFCH structure at various levels of earthquakes, 2) the city's upgrade proposal together with alternative upgrade options with a comparison of benefits and costs afforded by these upgrade options and an assessment of these options against FEMA program requirements, and 3) the estimated cost, based on schematic designs, of these upgrade options.

As part of the Task Force review, FEMA Office of General Counsel re-reviewed the issues surrounding the requirements of the building code "triggers," and its findings are presented here as part of the Task Force Report, together with the technical Panel's recommendations for funding of a building upgrade under FEMA's discretionary hazard mitigation program.

The members of the Task Force are:

Chair: **Craig S. Wingo**,
Director, FEMA Infrastructure Support Division

John P. Carey,
FEMA General Counsel

Technical Panel:
Robert D. Hanson
Professor of Civil Engineering, University of Michigan

Gregg E. Brandow
President, Brandow & Johnston, Structural Engineers, Los Angeles

Region IX Point of Contact:
Nicholas B. Nikas
Director of Response and Recovery Division

SECTION II:

FEMA REVIEW OF CODE REQUIREMENTS APPLICABLE TO THE SAN FRANCISCO CITY HALL

<u>A.</u> <u>SAN FRANCISCO BUILDING CODE (SFBC) ISSUES</u>

<u>Part 1: Legal Review</u>:

Until recently the City of San Francisco ("the City") has contended that Section 104 of the San Francisco Building Code (SFBC) triggers mandatory seismic upgrades to the City Hall in the wake of the Loma Prieta earthquake. The City's 1991 Repair Study, which presented a design for repairs to the City Hall, indicates the City's belief that the earthquake caused damages to the City Hall which exceed the damage threshold which appears at Section 104 of the 1989 SFBC and that seismic retrofitting of the City Hall is therefore required.

FEMA has agreed that the Section 104 of the SFBC applies to the post-earthquake repairs to City Hall. However, from an engineering perspective FEMA has disagreed that the earthquake caused sufficient damage to trigger the seismic upgrade provision which appears in the SFBC. Section 104(b) of the SFBC applies to the City Hall repairs. That provision states in pertinent part:

> *"B. <u>Structural Work</u>. The extent of an existing building that is considered as being directly affected by the new work, with regard to structural considerations, shall be determined using the following criteria:*
>
> *When structural alteration work is to be done on the roof or any floor of a building or structure, the work shall comply with the structural requirements of this code. The structural elements above and below the roof or floors involved shall be strengthened in accordance with the following criteria:*
>
> *(i) <u>Vertical loads</u>. When vertical loading is increased on the roof or floor of a building or structure, the roof or floor involved shall meet the structural requirements of this code and all structural elements*

supporting the roof or floor with the increased loading shall be capable of supporting the total imposed loads.

(ii) Lateral loads. When more than 30 percent, cumulative since the building was built, of the floor and roof areas of the building...have been or will be involved in substantial structural alteration and/or repair work the building...shall comply with Section 104(f). The areas to be counted towards the 30 percent shall be those areas tributary to the vertical load carrying components (joists, beams, columns, walls) that have been or will be removed, added or altered. (emphasis added)"

FEMA and the City agree that this language from Section 104(b)(B)(ii) of the SFBC mandates a seismic upgrade under Section 104(f) of the SFBC when more than 30 percent of the floor and roof areas of a building are being substantially structurally altered or repaired. However, FEMA and the City disagree on two other important issues. One disagreement involves the interpretation of Section 104(b)(B)(ii) of the SFBC, and the other disagreement involves whether, from an engineering perspective, the code trigger of the SFBC has been met. This portion of FEMA's appeal analysis is limited to a discussion of the first of these two disagreements - i.e., the interpretation of the meaning of Section 104(b)(B)(ii) of the SFBC.

The former Superintendent of the City's Bureau of Building Inspection issued a memorandum on February 4, 1991, which interpreted Section 104(b)(B)(ii) of the SFBC to require that areas to be counted in calculating the 30 percent code trigger of the SFBC should include both the vertical and the lateral load carrying components of a building. The City contends that the former Superintendent was authorized to make such "interpretations" of provisions of the SFBC and that the Superintendent's "interpretation" constitutes an "applicable code" or "standard" under subsection 406(e)(1) of the Stafford Act for which FEMA must provide funding under 44 CFR 206.226(b)(3).

FEMA disagrees with the City's interpretation of the impact of the February 4, 1991, memorandum of the former Superintendent. We base this conclusion on the clear language of Section 104(b)(B)(ii) of the SFBC. That provision mandates seismic upgrades of buildings based upon calculations of areas which are tributary to the vertical load carrying components of buildings. It would be incorrect to read into Section 104(b)(B)(ii) of the SFBC the words "or lateral" when those words could easily have been inserted into the ordinance when it was enacted. This is particularly true when the ordinance is structured in such a way that the vertical and the lateral load provisions of the ordinance are independent of one another. Section 104 of the SFBC was written primarily to deal with remodelling of buildings in general, not the simple repair of damage, and thus it is reasonable to find that the limitation to "vertical" was what was intended. Therefore, FEMA remains of the opinion that Section 104(b)(B)(ii) of the SFBC calls for the calculation only of areas which are tributary to the vertical load carrying components of a building in determining whether the seismic retrofitting mandate of the SFBC applies to a building's post-earthquake repairs.

On the question of whether the building official's interpretations can have a direct bearing on FEMA funding without FEMA control, FEMA General Counsel has made the following finding in connection with the role of the building official's discretionary decisions vis-a-vis FEMA funding:

> "The question of whether repairs require a complete seismic upgrade cannot be within the sole discretion of a local building official. This would wrest the authority to interpret and administer the Stafford Act and its implementing regulations from the Federal Government and place it in the hands of a local building official. This is particularly so when FEMA is charged with protecting the public fisc and the official is often an employee of the applicant." (Spence Perry, Acting GC to W. Medigovich, RD, Region 9, Feb, 12, 1993 p14)

<u>Part 2: Technical Review:</u>

In response to the city's position that the effects of damage (and thus the repairs) to the "areas tributary to the vertical load carrying components" is sufficient to trigger an upgrade under the SFBC, FEMA finds that this position is <u>not</u> supported by the information in the F/E Report or the city's second appeal brief. The Report for FEMA prepared by consultants Preece Goudie & Associates (PGA) states: *"Using the values given in the appeal...and applying them to the appropriate floor areas, the indicated area involved is 20% of the building area, far short of the 30% which would mandate seismic upgrade"* (p6). In addition, this report states: *"[the] capacity loss [in the city's appeal] is exaggerated"* (please see PGA Report, Appendix 8) PGA found that even the information presented in the F/E Report does not support the position that the code triggers are pulled.

Prior to the appointment of the Task Force, FEMA's consultants had also conducted their own independent survey of damage to one of the floors in order to verify the data submitted in the F/E Report. PGA reported that "they found that "*most of the floor cracks [were] quite old*", and whether re-cracking was "*due to the earthquake* *is problematic*." From this survey, PGA concluded that "*<u>F/E has greatly exaggerated the tributary area figures and that the actual results would place the cumulative area at 10%, or less...even further away from the 30% trigger</u>*." (Please see PGA Report, Appendix 8,)

Most of the floor cracks were found to be located in the floor slab directly above the beams. These cracks have been determined to be a result of the relief of stress at the top of the concrete slab after the building was constructed in 1910, rather than from the 1989 earthquake. The fact that movement occurred along these cracks during the earthquake does not indicate any particular distress or serious loss of strength. Since the cracks existed long before hand, such evidence of minor movement is normal, and cannot be prevented.

FEMA's consultants, PGA, found that the city's claim of sufficient damage to trigger the code was "greatly exaggerated" by Forell & Elsesser, Engineers (F/E) in their Report, and that even a conservative estimate of the real earthquake damage is far less than what was reported in support of the claim because many of the cracks were pre-existing and of little structural effect. In addition, the F/E Report has greatly understated the existing structural capacity of the San Francisco City Hall.

In addition, FEMA regulations require that for code upgrade work to be eligible, the code in effect must be "applied uniformly." A major portion of the city's claim is not only that an upgrade is required, but that the base isolation scheme is the most "cost-effective" of the alternatives which would meet the code. In evaluating this claim, one must look to see if any other buildings of similar type in the private sector are required to do upgrades meeting the same high standards which base isolation meets, or that there is no other less costly way to achieve less stringent code standards in the case of buildings of similar construction to City Hall. There is no evidence that private sector applicants for building permits to do upgrades to infill wall buildings have been required to carry out upgrades costing in the range of base isolation, which is $200 to $300 per square foot, or meeting its level of performance criteria. Since there are no such examples, a "uniform" application of the seismic requirements to other projects does not exist.

<u>B. STATE HISTORICAL BUILDING CODE (SHBC) ISSUES</u>

The City has contended alternatively that the seismic upgrade triggering mechanism of Section 104 of the SFBC does not apply to the City Hall. The City has acknowledged that the SHBC does not contain a "mathematical trigger" which mandates seismic upgrading of the City Hall. See page 16 of Rudolph Nothenberg's December 4, 1992, Second Appeal to Charles Wynne, the former Governor's

Authorized Representative for the Loma Prieta earthquake. The City nevertheless argues that several provisions of the SHBC, when read together, require the seismic upgrading of the City Hall in the aftermath of the Loma Prieta earthquake.

1. **Mandatory use issue:** The City contends - and FEMA agrees - that the City Hall is a qualified historic building. Section 8-103 of the SHBC provides:

> *The provisions of this code shall be applied for the rehabilitation...of qualified historical buildings....The SHBC shall apply when repairs...necessary for the preservation, restoration, rehabilitation, relocation or continued use of a qualified building...are made.*

FEMA does not dispute the applicability of the SHBC, but it is important to point out that the use of the SHBC is not mandatory on the building owner. The "mandatory" requirement is directed at the building officials of the cities and towns of California requiring that they approve projects designed under it, rather than at building owners requiring them to follow its provisions.

Since the SHBC is a more liberal and permissive code, pure economic self interest is expected to cause owners to use it because the amount of work necessary to meet it is less than is usually required to meet the Uniform Building Code. Because it is not mandatory on owners to use it, if the amount of work to meet its provisions is greater than that required by the "prevailing code," (such as, for example, if the SHBC had its own independent seismic upgrade "trigger"), then owners would automatically turn to the prevailing code for their building designs, rather than the SHBC.

2. **Minimum requirements issue:** The City also cites Sections 8-104 and 8-501 of the SHBC in support of its contention that the SHBC requires seismic retrofitting of the City Hall. Section 8-104 provides in pertinent part:

> *It is the intent of these regulations to provide means for the preservation of the historic value of qualified buildings...and, concurrently, to provide <u>reasonable safety from...seismic...forces</u>...for occupants of such buildings.* (emphasis added)

Section 8-501 of the SHBC provides:

> *It is the intent of this chapter to encourage the preservation of qualified historical buildings while providing a <u>reasonable level of structural safety</u> for occupants and the public at large through the application of this code.* (emphasis added)

FEMA does not disagree with the proposition that Sections 8-104 and 8-501 of the SHBC are relevant to the consideration of post-earthquake repairs to the City Hall. However, "reasonable" cannot be correctly construed as mandating that the potential for building damage be eliminated through the use of extraordinary and excessively costly means. This would be inconsistent with the provision in the same paragraph which provides:

> *"It is not the intent to protect the property and by so doing adversely affect the historical integrity of the structure."*

The SHBC puts a priority on leaving things alone, even if a future seismic event may damage the historic fabric of a building.

3. **Upgrade "trigger" issue:** The City goes on to contend that, as a result of the Loma Prieta earthquake the City Hall is a "substandard" building, as that term is used in Section 8-109 of the SHBC. That provision of the SHBC states:

> *Any qualified building that does not comply with the minimum conditions (of the SHBC) may be considered substandard and must be corrected as a condition of the use of (the SHBC).* (emphasis added)

The City's contention that the City Hall is "substandard" is based upon its belief that the Loma Prieta earthquake caused substantial structural damage to the building. The City contends that in the aftermath of the earthquake the City Hall does not contain a "complete, continuous and adequate stress path", as required by Section 8-505 of the SHBC. Section 8-505 of the SHBC provides in pertinent part:

> *The application of alternative structural regulations where provided for by this chapter shall comply with the following procedures...(3) The capability of the structure to carry lateral forces shall be evaluated. <u>A complete,</u> continuous and adequate stress path, including connections, from every part or portion of the structure to the ground <u>shall be provided</u> for the required horizontal forces.* (emphasis added)

The city takes the position that this code language mandates an upgrade because the city's consulting engineer has taken the position that the required horizontal forces are those stipulated for new buildings in the prevailing code (100% UBC) and that the stress path is "inadequate" for these forces.

FEMA's position is that the SHBC does not stipulate that the *"required horizontal forces"* be interpreted as requiring a seismic upgrade to the force levels in the *"prevailing code"* if no upgrading is otherwise mandated by provisions in the prevailing code or local ordinances. Proof of this intention is provided by the next provision (8-505.4) of the SHBC where the SHBC does stipulate for parapets, that upgrading to the prevailing code force provisions must be carried out, even if no pathology is observed. If the drafters had intended that the provision cited above regarding the rest of the structure mandated an upgrade, they would have so stated.

In the Yuma Building case, the State Historical Building Safety Board decision states that "*the SHBC excludes "trigger mechanisms" that require comprehensive upgrading.*" The interpretation of the language of the SHBC as having independent "triggers" is contrary to the stated intent of the SHBC, which is to allow for and encourage the use of more economical and less destructive solutions than would be allowed under the prevailing codes. If the SHBC were indeed to be interpreted as having its own independent trigger, this would lead to a situation where two identical buildings, one designated as historic, and the other not, would be treated differently when identical damage is being repaired. The historic one would be subjected to a much more costly and invasive repair and upgrade scheme than the identical non-historic one.

It is FEMA's position that, since no code required seismic upgrade has been triggered, the analysis of the "*stress path*" as specified under the SHBC for the SFCH is intended to be a measure of it against the "*reasonable safety*" criteria stated in 8-104. The "*minimum conditions*" of the SHBC referred to in the definition of "*substandard building*" is limited to life safety risks which fail to meet the "*reasonable*" criteria discussed above. In the event that a unreasonable risk is found to exist in a building, the intent of this provision of the code is only to require specific repairs to correct the identified risk. This provision was not intended to be a "trigger" for the upgrading of the building as a whole.

In the case of San Francisco City Hall, the city has repeatedly declared that the building is safe for occupancy. The City has occupied the building continuously for 4 years, and has posted signs for a period immediately following the earthquake informing the public that the building was not dangerous.

4. **Occupancy issue:** The City argues that the City Hall is a "*critical occupancy*" building pursuant to Section 8-502 of the SHBC because it contains public assemblies of over 300 occupants.

Section 8-502 of the SHBC provides in pertinent part:

> *"The alternative structural regulations provided by Section 8-505 shall be applied in lieu of or in combination with any prevailing code provisions, to the...repair...of any qualified historical building....The foregoing shall not be*

construed to allow the enforcing agency to approve or permit a lower level of safety of design and construction than that contemplated by the prevailing code provisions in occupancies which are critical to the safety and welfare of the public at large including, but not limited to...public assemblies for over 300 occupants."

As a result, the City contends, Section 8-502 of the SHBC requires that the City Hall be repaired using an engineering design which will provide a level of safety which is at least equivalent to the level "contemplated by the prevailing code provisions" of the SFBC. The City contends that the "prevailing code provisions" are those specified for new buildings which appear in Chapter 23 of the SFBC.

The City points out that the SFBC imposes an engineering design requirement that code conforming seismic upgrades of existing buildings must normally be performed in such a way that the building will have 75 percent of the full Uniform Building Code (UBC) force levels after the seismic upgrade is finished. With respect to the City Hall, however, the City argues that Section 8-502 of the SHBC requires a higher performance level than that imposed by the SFBC. The City notes that:

...the City's consultant has concluded that the requirements necessary to resist 100% of UBC/SFBC seismic base shear for critical occupancies are applicable and appropriate for (the City Hall). (See page 5 of John Roddy's February 17, 1994, analysis of the SHBC.)

It is FEMA's position that by not allowing a "*lower level of safety...than that contemplated by the prevailing code provisions..." the SHBC does not intend that historic "schools, hospitals and assembly buildings*" be subjected to more stringent seismic provisions under the SHBC than would be required of non-historic buildings of the same type under the "prevailing code" alone. Therefore, even if the City Hall is seismically upgraded (which FEMA has determined is not required by code), the design force specified by the SHBC for an "assembly building" is no higher that that required for existing assembly buildings in the SFBC, namely approximately 75 percent of UBC values as cited above.

<u>Summary on SHBC Issues:</u>

In summary, the City's contention about the applicability of the SHBC is based upon their determination that the building is currently "substandard." This analysis, however, ignores the fact that Section 8-104 of the SHBC only contemplates repairs which provide reasonable safety from seismic forces." This provision is limited to life safety concerns and does not mandate repairs which would prevent non-life threatening damage to the historic building itself.

From an engineering perspective the Panel has determined that the Loma Prieta earthquake did not cause sufficient damage to the City Hall to make the building significantly more dangerous than it was prior to the earthquake.

GENERAL CONCLUSION

Although the City Hall does not meet the legal requirements of sufficient damage of Section 104 of the SFBC or the definition of "substandard" in the terms of the SHBC to mandate a seismic upgrade, the Panel recommends that FEMA provide funding under its discretionary hazard mitigation program for a life safety upgrade of the building. Under this program, funds can be provided for cost-effective upgrading of structures against future potential natural disaster hazards.

SECTION III

ASSESSMENT OF THE SAN FRANCISCO CITY HALL STRUCTURE AND THE BENEFIT/COST DATA

A. Engineering Review of the <u>Repaired</u> (but not upgraded) City Hall Building Earthquake reponse

INTRODUCTION

San Francisco City Hall was constructed after the 1906 Earthquake using the technology and experience on hand at that time. A steel skeleton with a brick exterior was the choice for most major structures, and the performance of this class of buildings has been good in the recent earthquakes. The stability of the steel frame when combined with the stiff, energy absorbing unreinforced masonry walls is still recognized to be earthquake resistant.

San Francisco City Hall was designed to have a soft story at the Main Floor, which was intended to absorb the earthquake's energy at the lower level in the building so that earthquake forces would not be amplified to destructive levels at the upper floors and the dome. Brittle materials such as hollow tile walls were used, rather than stronger brick masonry to insure that the building could sway at this level after the initial shock of the earthquake had damaged the hollow tile. The concept of a soft story is a viable dynamic solution to the earthquake problem if the building can be assured of remaining stable, and the degree of damage is acceptable. Base Isolation is the soft story concept carried out in a more technologically sophisticated way.

<u>1. Forell/Elsesser Position:</u>

In January and February 1994 Forell/Elsesser provided their assessment of the performance characteristics of the "original" SFCH building and its likely performance for different earthquake ground motion intensities. They selected the North/South direction of the Main floor level for their analyses. Damage predictions for the dome, including any potential for collapse of the dome alone, were considered separately.

The F/E assessment was presented in the form of analyzing building shear capacity at the main to second floor level as a function of interfloor displacement. The major contributors to the lateral capacity in this assessment were the masonry walls at the entrances and the corners of the building, the slender masonry piers between the windows, the steel frames, and the hollow clay tile. From their assessment of each element of the structural system taken independently, the masonry walls at the entrances and the corners of the building have a capacity of about 0.04g to 0.07g, the slender masonry piers a capacity of about 0.01g, the steel frames a capacity of about 0.01g to 0.02g, and the hollow clay tile walls a capacity of about 0.03g to 0.04g, (see Figure III.1). These maximum capacities occur at different interfloor displacements so these maximum values cannot be added directly. Nevertheless, these values do overlap, so that the building's overall maximum capacity is substantially greater than any of these numbers taken individually. From Forell/Elsesser's data, the combined value which shows the building's maximum strength is about .11g.

Based on their analysis, Forell/Elsesser have taken the position that the SFCH would collapse at the Main Floor level at an earthquake Modified Mercalli Intensity level (MMI) of VIII (an effective peak ground acceleration of 0.16g to 0.32g.) A significant part of their analyses which leads to this conclusion was their finding that the structural steel frame (treated as a bare frame without the masonry infill) was capable of supporting the static building weight only to a drift displacement of about 2% of the story height (about 5 inches).

2. The Task Force Findings:

The Task Force Panel (Panel) elected to evaluate the lateral capacity of the Main floor level in the North/South direction for comparison with the Forell/Elsesser results. In order to evaluate its potential impact on the Forell/Elsesser damage assessments for various earthquake intensity scenarios in the benefit/cost analyses, only the capacity of the steel frame of the SFCH has been studied in detail by the Panel.

The Forell/Elsesser assessment assumes that the double angle beam web connections to the columns will yield, the column just above the main floor will yield, and that the strength will degrade due to rivet fracture at the built-up column and connections.

The F/E calculations have been used as the basis for the Panel's evaluation of the strength of the steel frame. The following assumptions establish what the Panel believes are <u>lower</u> bound estimates of the Main floor steel frame's seismic capacity.

1. The concrete fireproofing of the steel (full section) is included in the weights and masses but is excluded from the strength and stiffness evaluations by F/E and the Panel. In actual fact, this fireproof jacketing of all of the beams and

columns would improve the seismic capacity of the frame on the order of 10%.

2. A nominal steel yield stress of 33 ksi was used without consideration of strain hardening or actual steel yield properties in excess of specified minimum values. Forell/Elsesser used yield strengths of 30 ksi and 60 ksi to provide lower and upper bounds to their steel strength estimate.

3. Test data from the University of Illinois on flexible riveted connections were used by the Panel to establish the probable strength and stiffness of the beam to column strong axis connection into the deformation ranges envisaged during the earthquake response.

4. Only strong axis column contributions were included in the lateral stiffness and strength calculations. It was estimated by the Panel that the contribution of the weak axis column participation would be small.

5. In the assessment of lateral capacity by Forell/Elsesser and by the Panel, the exterior stone walls are ignored except as they contribute to the seismic mass of the building.

   When the height of the main floor level decreases slightly as a result of large story drift, the massive stone and brick wall would be compressed as well as forced to deform and crack. The massive stone blocks which cover the exterior of the building, together with the multi-wythe brick wall attached behind, would be caused to rub against each other. This friction will result in a significant increase in energy dissipation. In addition to the energy dissipation provided by these walls, the lateral capacity of the masonry walls extends beyond the elastic shear capacity of the masonry itself because the tremendous compressive strength of the masonry would still be engaged even after the walls begin to crack.

   Although it was not possible for the Panel to assess accurately the contribution of these exterior walls to the seismic performance of SFCH in the time available, these walls will significantly help decrease lateral displacements and increase earthquake energy dissipation in the building as a whole. By ignoring this phenomenon, both Forell/Elsesser's and the Panel's predictions of story deflection and building damage are conservative compared to what is likely to actually occur.

The Panel's calculation of the behavior of the steel frames in the main to second floor level indicates that the primary shear capacity develops from the yielding of the column just above the main floor and just below the second floor with a capacity of about 0.05 g for the lower bound. This behavior requires a relatively intact wall system at the second floor level when the hollow clay tile walls fracture in the main floor level. If the behavior of the second floor walls is not stiff enough, the top of the column yielding will transfer to the column splice located eighteen inches above the second floor and will mobilize the beam to column connections. This mechanism will give a lower bound capacity of about 0.04g. These lower bound capacities will be reached at about 3 inch displacement, and will retain this capacity until the displacements are about 6". The Panel's calculations are provided in Appendix 1.

The panel has concluded that the structural steel framing system in the Main floor level N/S direction, treated as the lower bound of the bare frame alone, is about 2 1/2 times stronger at an interstory deflection of three to six inches than described by Forell/Elsesser. These capacity and deflection calculations assume that the entire weight of the dome and building above the Main floor level is carried by these steel frames, rather than any vertical support provided by the existing masonry walls.

In order to estimate the earthquake response of the SFCH to various future MMI levels, with the corresponding damage and repair costs, the Panel used (1) information from the F/E Report and 1994 assessment data, (2) information gained from the Panel's review of the original 1913 drawings of the SFCH, (3) the Panel's observations of the damage to the building from the Loma Prieta Earthquake, (4) the Panel's analysis of the dynamic characteristics of the building, and (5) the Panel's personal observations of earthquake damage to buildings in general. These estimates are summarized in the next section.

B. Benefit/Cost Data.[1]

The Panel concludes that the Forell/Elsesser damage estimates for the repaired, but un-upgraded, City Hall building are high for each of the lower intensity earthquake conditions. The following provides the Panel's assessment of the building performance and associated estimate of the repair costs.

Since the Loma Prieta earthquake (an event which falls within MMI level VII) provides a real data point for damage and repair costs, this earthquake record was used as the starting point for the assessment and cost estimations. Loma Prieta had a

[1] It should be noted that the Panel considers that the benefit/cost analysis can provide relative comparison numbers which should not be interpreted in absolute terms. The Panel recommends that the dollar amounts related to a given benefit/cost percentage shown in the benefit/cost model results not be used as a benchmark for determining FEMA funding amounts.

local peak ground acceleration estimated to be about 0.10g and resulted in a repair cost adjusted to 1994 of approximately $15 million. By decreasing and increasing damage and repair costs within this range of accelerations a repair cost of $12 million was estimated for 0.08g and $30 million for 0.16g ground motion accelerations.

For 0.16g shaking (upper bound for MMI VII and lower bound for MMI VIII), it is expected that (1) some of the hollow clay tile walls will need to be replaced, (2) widespread repairs of other hollow clay tile walls will be required, and (3) widespread repair of damage the exterior masonry, especially in the Main floor level will be needed.

At 0.32g ground motion (upper bound for MMI VIII and lower bound for MMI IX), the major damage at the Main floor level will require (1) replacement of all of the hollow clay tile walls, and replastering of most of the wall and ceiling finishes, (2) the dismantling and resetting of displaced exterior and interior stone cladding with the grouting of cracked underlaying brick work at the building corners and either side of the main entrances, and (3) the quarrying and cutting of some replacement stone. It is also anticipated that some steel braces in the dome will buckle, and that some permanent offset in the Main floor framing will occur. The repair cost of the foregoing is estimated to be in the range of $120 million.

At approximately 0.36g ground motion, it is estimated, for the purposes of the benefit/cost model, that the building damage will equal the replacement cost of $200 million. This replacement cost of approximately $400 per square foot represents the cost of new construction for a major new monumental city hall building of similar presence and character to the present building. For the purposes of this benefit/cost model, this figure is the cap to the damage estimates based on the expectation that costs of repair beyond this level would result in the replacement of the building.

<u>While earthquakes of MMI X and above</u> have never been known to occur historically at the site, they are included for statistical completeness at the replacement cost of $200 million. Data for these level events will have little effect on the benefit/cost results.

The Task Force has taken the position that no further amount should be included beyond this $200 million to represent the loss of the historical building. Since there is not an established policy on the procedure to attribute historical value to destroyed historical existing buildings, the Panel has not included such a value in their input data. [2]

[2] It should be noted that, in the case of Oakland City Hall, a base isolation project currently under construction, FEMA funding was capped at 75% share of the $50 million cost of a new building when the building had been determined to be over

Damage to the contents has been estimated by the panel in rough dollar amounts, based on the total value for the contents of $84 million provided by the city. The death rate is based on ATC 13.

(Please see Appendix 2 for details on the Benefit/Cost Model and input data for this project.)

50% damaged. The cost of the base isolation project, including renovation of the building is approximately $76 million.

SECTION IV

THE FEDERAL DESIGN AND PROJECT COST OPTIONS

A. INTRODUCTION

Although the City Hall does not meet the legal requirements of sufficient damage of Section 104 of the SFBC or the definition of "substandard" in the terms of the SHBC to mandate seismic upgrade, the Panel recognizes that the original building's seismic design concept has several weaknesses which could become life-safety issues as the ground motion intensities increase beyond those anticipated in the original design.

As concluded in Section II, it seems prudent to voluntarily implement cost effective mitigation measures for the San Francisco City Hall to increase life safety beyond its current level. It is known that different mitigation measures will result in different building performance characteristics, and that the corresponding mitigation costs will be different. The following portions of this report will focus on two potential mitigation schemes, as well as the City's base isolation scheme, with their degrees of seismic protection and their relative costs. This discussion will lead to recommendations for actions by FEMA.

After reviewing the data presented by the applicant, the Task Force decided to include the conceptual design and cost evaluation of two new options for the seismic upgrading of the San Francisco City Hall. This has been done because alternative schemes provided by the City did not consider different seismic performance levels. The alternative options have been developed to establish the feasibility of seismic designs at less cost which conform to the Federal program goals for Public Assistance disaster relief funding for hazard mitigation under Section 406 of the Stafford Act, and 44 CFR 206.226(c) in the FEMA Regulations.

These designs have been developed to a conceptual design level, similar to that intended as a FEMA funding review benchmark within the recent Northridge Earthquake agreement (DR-1008-CA). Although the designs are conceptual, the Panel has concluded that these designs when carried to final details could be executed for roughly the costs shown here. It is intended that these designs would meet or exceed the 1989 San Francisco Building Code, Section 2313 forces for the seismic upgrade of an existing building.

FEMA Section 406 Grant Program Eligibility Criteria:

In this report, first, the performance objectives which the Task Force has determined are suitable for a FEMA hazard mitigation funded upgrade of the SFCH will be described. Second, two different alternative seismic upgrade design options, Federal Design Options A & B, will be presented.

FEMA regulations for the repair, retrofit, upgrade and risk mitigation for seismic disasters can be stated in its simplest form as follows:

a) The repair of earthquake damaged buildings to their pre-earthquake seismic resistance capabilities is eligible for FEMA funding.

b) If the damage is so severe that the applicable building codes or local ordinances in effect at the time of the initial project approval date require the strengthening of the building in its entirety, then the costs to retrofit the building to current seismic codes applicable to existing buildings is eligible.

c) If a hazard mitigation measure is shown to be cost-effective, FEMA, in its discretion, may consider making costs of such measures eligible for FEMA funding. (In this case, since it is not required by code as a part of the repair of the earthquake damage, the seismic upgrading of the building would be the hazard mitigation project.)[3]

As a result of the findings stated in Section II of this report, the San Francisco City Hall falls into item (c) above. The damage caused by the Loma Prieta Earthquake has been determined in three separate reviews by FEMA and its professional consultants not to be severe enough for an upgrade of the building to be required under the applicable building codes.

[3] An applicant may elect to pursue a different plan, as long as it meets the same hazard mitigation objectives, at a greater cost. In such a case, the project is declared an "improved project" under FEMA regulations, and the Federal funds are capped at the level which FEMA determines is necessary to meet its objectives and regulatory limits.

Task Force FEMA Funding Alternatives - Federal Design Options

For this report, The Task Force has reviewed the merits of providing funding for the City Hall under the FEMA discretionary program for hazard mitigation. FEMA regulations limit funds to hazard mitigation work which can be shown as cost-effective. As a result, in order to properly judge the appropriateness of any one proposed upgrade design, the Task Force found that it needed to explore potential alternative schemes with different performance objectives and different costs for comparison with the City's base-isolation design.

At the outset of the Task Force's review, only two schemes were available for comparison: 1) base-isolation with extensive superstructure strengthening (the City's preferred option), and 2) a fixed base design with extensive poured-in-place new shearwalls extending up through the building. Both of these schemes were designed by Forell/Elsesser Associates, Engineers, and both were estimated to be of approximately the same cost ($110-120 million plus design costs, plus relocation costs for 3 years).

The Task Force determined that these alternatives provided insufficient basis for an evaluation of cost-effective options for potential FEMA discretionary funding. Thus, the Panel considered other potential upgrade schemes which would fit within a wider range of performance objectives to determine if the FEMA cost-effective criteria could be met.

The City's and the Federal Performance Criteria Compared

City's Criteria **(From F/E Report pE-6)**	**FEMA (Task Force) Criteria**
1. *Attain modern life-safety standards with regard to both the primary structural and non-structural elements.*	1. Attain life-safety consistent with SHBC at SFBC Section 2313 force levels for seismic mitigation.
2. Provide complete load-paths, without discontinuities, for earthquake induced forces.	2. Strengthen weaknesses in load paths where required to achieve life safety criteria.
3. Provide protection for historic elements from future earthquake damage.	3. Reduce, but not prevent, damage to historic finishes in future earthquakes consistent with life-safety objectives.
4. Reduce to a minimum the impact of the repair and strengthening program on the building's historical interiors and exterior facade.	4. Preserve historic fabric within the limits of cost effective seismic upgrade design, and in conformance with the Secretary of the Interior's

	Standards for Rehabilitation.
5. Minimize disruption to the occupants.	5. (same)
6. Provide a cost effective repair solution.	6. (same)

Table IV.1: Seismic Performance Criteria

The City has explained to the Task Force that their performance objective which supports their decision to use the base-isolation design is to minimize or eliminate building damage for a design level earthquake (475 year return period) in order to provide protection of the building's architectural details beyond that which is possible with a fixed-base design.

Under its congressional mandate, FEMA necessarily has more limited objectives. The Federal performance criteria is to reduce the potential for damage in order to reduce life-safety risks. The protection of the property is a Federal goal, even for historical buildings, only if it can be demonstrated to be cost-effective. The differences between the City's upgrade design and the Federal Options is due to this difference in performance objectives.

Building Code Compliance:

The Task Force has reviewed the code requirements which apply to the seismic upgrade of existing buildings in San Francisco, and has determined that the design criteria for the Federal Design Options should meet the seismic force levels specified in Chapter 2313 of the San Francisco Building Code. Since the San Francisco City Hall is not classified as an "essential facility" under the building code (which applies to police and fire stations, hospitals, emergency vehicle garages, and the like, not city halls,)[4] the meeting of the seismic force levels of Section 2313 of the SFBC would satisfy the requirements of the State Historical Building Code (SHBC) for full current code upgrade of this structure. (See Section II of this Report for full explanation of applicable codes)

As will be explained, for the Federal Option A Design, the El Centro N/S accelerogram, with its acceleration intensity increased to achieve a linear elastic response spectrum 1g maximum level was used. This motion exceeds the code level

[4] The applicant has not claimed that the San Francisco City Hall is an "essential facility" under the SFBC, so this is not an issue in the dispute.

force by approximately 20% for the period range of concern for the building.

Seismic Strengthening Alternatives

The recently identified problems with San Francisco City Hall mostly relate to the building's seismic design deficiencies when measured against current engineering standards. Present day seismic design is based on a better understanding of the seismic ground shaking and building response. Engineering practice now has the capacity to analyze complex structures mathematically, supplementing engineering practice based on judgement and observed performance.

The structural system of the building can be quantified. The structural system of the City Hall, measured by the standards of today, shows that it does not have the strength, ductility or control of architectural damage that is mandated by current building codes. The stability of the 20 foot high Main Floor, and the lateral capacity of the dome structure are the primary concerns with the City Hall structure. Damage to the architectural skin and interior wall surfaces, resulting from excessive deflections in these weaker areas, is a secondary concern.

If life safety was the only goal of seismic strengthening of the City Hall, the building stability against collapse would be a sufficient level of seismic strengthening. If damage control is desired, increased seismic strengthening is what is typically required.

Controlling falling hazards is also a life safety goal, but the number of potential lives lost from falling objects is substantially less than are at risk if the building collapsed. Falling hazard risk can be mitigated by either reducing building deflections by strengthening or by reducing its seismic response by either dissipators or base isolation. It can also be dealt with by installing a separate schedule of secondary anchors.

The seismic strengthening scheme for a project such as San Francisco City Hall must be developed based on the goals appropriate for the building. Descriptions of schemes with increasing levels of performance are summarized below to illustrate the range of mitigation measures that could be applied to the building:

1) No Upgrade,
Repair Damage Only: The building historically has performed well in small to moderate earthquakes. Potential severe damage or collapse has been postulated for a very large earthquake.

2) Strengthened Building Design: (Federal Option B)

Add strength to existing brick walls and strengthen dome. Provide stability for a large earthquake. Damage is controlled for moderate earthquakes and potential for collapse is prevented. Extensive damage may still be possible in a very large event.

3) Energy Dissipation Design: (Federal Option A)

Reduce overall building motions with mechanical energy dissipators. Assures stability but accepts a controlled amount of damage.

4) New Shear Walls Design: (City Alternative B)

Add new reinforced concrete walls of sufficient strength to resist the expected seismic forces while decreasing deformations and increasing strength and damage control in comparison with approach 2.

5) Base Isolation with upper story strengthening Design: (City Alternative A)

Insert base isolator bearings beneath the building, and strengthen building above isolators. The isolation system would reduce the seismic response and the strengthening would avoid "tuning" of isolator and building periods. Provides maximum protection of the brittle architectural finishes of the building.

Seismic strengthening ordinances have typically focused on the minimum level to provide reasonable life safety rather than more extensive damage control. Generally the costs rise as more damage control is sought, while this is not absolutely the case. As is shown by the Federal Options, because of the unique configuration and dynamics of the City Hall, the use of superstructure dampers is expected to afford a greater degree of life-safety protection and overall damage control at less cost than the more conventional perimeter and courtyard wall strengthening design in the larger earthquakes.

B. FEDERAL DESIGN OPTION A: SUPPLEMENTAL ENERGY DISSIPATION SYSTEM WITH DOME, DOME TRANSFER, MAIN FLOOR, GROUND FLOOR STRENGTHENING

When the City Hall was constructed in 1912, the original engineer, Christopher H. Snyder, included what he called a "flexible story" in its design. This concept was executed at the Main floor level of the building by replacing the strong masonry interior walls between the main floor slab and the second floor slab with weaker hollow clay tile walls. The exterior walls of the building were built as full masonry walls, but the masonry walls of the courtyard exist only above the second floor slab level. The steel frame piers which support the main transfer girders under the dome are braced with both steel and concrete, except at the main floor level where they are clad only in hollow tile.

The intention of this design was to deliberately concentrate the deflections caused by an earthquake at this low level in the building, and thus reduce the potential for severe amplification of forces in the upper stories and the dome where the resulting damage could be severe and life-threatening. A 1929 report on the system devised by Snyder reports:

> *"The flexible theory contemplates that the stress shall be absorbed or taken up by the deflection of the frame, the walls in the flexible [main floor] portion of the building being designed to permit such deflection without damage or to crush with the thought of economical repairs, the damage being confined to this zone."*[5]

The original design relies on the cracking of the exterior masonry walls and the interior hollow clay tile walls in the main story level for the desired reduction of the top floor and dome amplification. This design envisages energy dissipation in the main floor by the damage to hollow clay tile walls, while further reducing anticipated destructive amplification higher up.

Although it was the only means available at the time, the drawback of this design is that, with larger earthquakes which have more recently been predicted, more damage can be induced at the main floor level than may be acceptable or even accurately predictable. The city's base-isolation design solves this problem by isolating the structure at the basement level, strengthening all stories, including the main floor, with new shear walls extending up to the roof, and putting new bracing in the dome. The Federal Design

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[5] Earthquakes and Building Construction, Clay Products Institute of California, 1929

Option A is based on utilizing and augmenting the benefits of the original design feature, while restraining the story drift at the Main level to acceptable and predictable levels, with a commensurate reduction in the potential for damage at that level.

In addition to providing restraint, Federal Design Option A introduces energy dissipation at the Main floor level with the use of mechanical energy dissipation devices. These devices will have the effect on the building much the same as shock absorbers have on a car. The advantage of this is that a known and predictable amount of energy dissipation can be introduced at comparatively low amounts of story displacement ("drift"). As a result, the level of inelastic deformation within the masonry and hollow clay tile walls of the building can be kept within predictable and acceptable limits, reducing or eliminating the life-safety risks, as well as the likelihood that the future repair costs will be excessive.

Described simply, this approach is designed to <u>eat up energy</u>, rather than <u>isolate energy</u>, as with base-isolation. The energy dissipation devices will work in tandem with the best features of the building's original structural system, thus reducing the costs necessary to gain a substantial reduction in life safety risk and extreme property damage while improving the building performance over that of a shearwall design.

1. <u>Energy Dissipating Devices</u>

Supplemental energy dissipation devices of many types are manufactured and have been installed in both new and retrofitted buildings in the United States, Canada, Mexico, Japan and New Zealand to mention a few key countries over the past 25 years. They are currently used for severe wind storm building vibration control as well as seismic force and deformation reduction. Over these years, the primary use for supplemental damping systems has been the reduction of motion from wind, but in recent years, extensive research on their application for seismic design has been conducted, and a number of installations have been made, including several retrofit projects.

Supplemental energy dissipation technology is different in philosophy from base isolation, even though base isolation systems need to utilize energy dissipation devices. The basic concept of base isolation is to decrease the amount of energy transmitted from the ground to the building by shifting the fundamental natural frequency of the building to a frequency which has relatively little earthquake energy. Thus, the earthquake energy collected by the building is smaller than for the corresponding "fixed base" building. Supplemental energy dissipation systems are designed to dissipate the earthquake energy input which otherwise would be stored in the building to destructive levels, rather than shift the buildings's natural frequencies.

The devices have been developed with different mechanical properties using different energy absorbing materials. The most common devices utilize viscous materials (like a car shock absorber), viscoelastic materials, metallic yield dissipation, and friction. Buildings using these types of devices have been constructed or retrofitted in California, Canada, Japan, Mexico, and New Zealand.

The Applied Technology Council (ATC) held their first seminar/workshop in 1986 on the topic of both Base Isolation and Supplemental Damping Systems (ATC 17). In 1993 ATC held a update seminar/workshop to discuss Base Isolation, Supplemental Energy Dissipation, and Active Control Systems (ATC 17-1). Although supplemental energy dissipation device applications are relatively new compared with standard fixed-base steel and reinforced concrete construction technology, they are very similar in technological development and application as base isolation. (Please see the paper <u>*State of the Art, State of the Practice in Base Isolation*</u>, by James Kelly, Appendix 6)

2. <u>Description of the Design:</u>

In order to make the conceptual designs "generic", linear viscous dampers were assumed to be the energy dissipation devices. It will be relatively straight forward to design and implement viscoelastic, metallic yield, or friction devices once the desired viscous characteristics have been established.

The original seismic design planned for a soft/weak story characteristic at the Main floor level. This is a logical location for the introduction of supplemental energy dissipation devices. In discussions with the Forell/Elsesser design team it was made clear that the one of the main problems they observed with the existing SFCH structural system is the whip action of the dome as excited by the building response. This suggests that the dome and dome-to-building transfer regions are also a logical location for supplemental energy dissipation devices.

The conceptual design considered using c = 100 kip-sec/inch between the roof and the octagon deck, the octagon deck and the drum, and the drum and the upper dome; and 2000 kip-sec/inch between the Main floor and the second floor. The dynamic building response results will be described next, with details provided in Appendix 3.

Table IV.2 summarizes the elastic maximum dynamic responses of this system subjected to a modified 1940 El Centro N/S accelerogram representative of the 1988 UBC design spectra. The maximum relative story displacements are given in column (2), the maximum elastic story shear forces in column (3), the existing story capacities provided by Forell/Elsesser in their Table 8.1 (revised 1/94) in column (4), the ratio of demand to capacity which represents ductility demand in column (5), the estimated maximum elastic floor accelerations in column (6), and the reduced floor accelerations due to inelastic action in column (7), and the base isolation accelerations in column (8). Where data is not available, the table is left blank.

(1) Level	(2) Max. Rel. Displ. (inch)	(3) Max. story shear	(4) Exist. Capac.	(5) Strength Ratio	(6) Elastic Accel. (g)	(7) Reduce Accel. (g)	(8) Base Isolate Accel. (g)
Lantern	0.06	75			0.53	0.26	
Dome	0.22	324			0.36	0.18	0.23
Drum	0.50	4,497			0.45	0.22	0.22
Octagon	0.90	8,035			0.54	0.27	0.19
Roof	0.19	8,870	14,600	0.61	0.22	0.11	0.18
4th Floor	0.34	19,508	18,400	1.06	0.28	0.14	0.18
3rd Floor	0.69	26,381	13,300	1.98	0.28	0.14	0.17
2nd Floor	0.76	19,534	12,900	1.51	0.15	0.10	0.17
Main Floor	0.22	43,952	61,200	0.72	0.27	0.27	0.17
Grnd Floor					0.34	0.34	0.17

Table IV.2: Federal Design Option A-Dynamic Response Results and Comparisons

The maximum relative story displacements and the maximum story shear forces are direct computations from the time history analyses. The existing building capacities have been provided by Forell/Elsesser as a revised Table 8.1 from their report. The capacities of the HCT have been excluded from the values in Table IV.2. The ratio of elastic demand to existing capacity indicates the level of inelastic action anticipated by this selected ground motion. It can be seen that the maximum ratio is about two between the second and third floor levels.

Different ground motion accelerograms will give slightly different results. To estimate the inelastic accelerations of the building, the elastic accelerations are divided by the anticipated maximum ductility ratio below that level. It can be seen that the building, without additional strengthening, will have a ductility of about two above the second floor. Some strengthening could reduce this inelastic demand, but would increase the building and dome accelerations above the reduced levels in column (7) of Table IV.2. Addition of energy dissipation devices above the second floor could reduce the inelastic demand without increasing acceleration.

In the event of an earthquake of extremely large magnitude, the energy dissipation support structures are designed to help carry the weight of the building. The restraint provided by the energy dissipation devices and their supports would maintain a story displacement (drift) within a range of plus or minus three inches, which will limit damage to the masonry at that floor level. As the existing masonry begins to displace slightly, increasing amounts of energy will be transferred to the new energy dissipation devices. This removes energy from the structure, and reduces the shaking response. Reduced displacements will reduce life-safety hazard and extensive damage to the architectural details.

As reported in Chapter 9 of the F/E Report, the initial analysis of the building by the F/E team did not show a good correlation between the observed damage from the Loma Prieta Earthquake and the linear elastic analysis of the structure. "*Based on the stresses alone, much more damage would have been expected."* (p9-15) When allowance was made for the "softening" due to earthquake damage, *"the fundamental period of vibration lengthened...from T=1.0 to T=1.3 seconds."* This *"shift in building period due to the damage to the walls in the main floor of the building caused a 70% reduction in the dome seismic forces. This reduction brought the dome forces down to about 0.17g (from 0.25-0.28g)"* This resulted in a *"good correlation between the calculated and observed damage based on the force level."*

This analysis illustrates the benefits of building "softening" documented in the Forell/Elsesser analysis. Their concerns were that the masonry would gradually lose strength, as well as stiffness. The Federal Option A, with the insertion of the mechanical energy dissipators would preserve and make use of the beneficial effects of the reduction of seismic forces resulting from the lengthening of the building period, while reducing the damage to the masonry walls and preventing danger of building collapse.

<u>Dome Drum and Dome Transfer Area</u>: In addition to the energy dissipating devices located at the Main Floor level, Option A also includes energy dissipation devices at the dome transfer and dome drum. These energy dissipation devices would be installed into a new structural bracing system similar to that currently proposed as part of the Forell/Elsesser design at the dome transfer and drum levels. By installing energy dissipation devices into these locations, the strengthening of the dome/drum area will be accomplished without significantly increasing the stiffness. The energy dissipation also controls the dome vibration.

<u>Energy Dissipation Devices</u>: It should be remembered that the Federal Design Option A is based on the use of generic viscous damping values attributable to a number of different device types, rather than a specific device. If this design were taken through the design development stage, the type of device for each different area of installation would be defined, but the values used here are intended only to establish the efficacy of this approach. Appropriate supplemental damping devices with proven response characteristics can be selected easily.

Construction Considerations: The supplemental energy dissipation building scheme has the following major components:

1. Supplemental energy dissipation devices are connected between the lower flange of the beams of the second floor and the top of added braced frames or reinforced concrete walls independent of the Main floor framing system with a gap of two inches on each side. The selection of braced frames or reinforced concrete walls depends upon whether the Main floor location is in partition walls or in office space. The second floor beams would be supplemented with steel T sections for shear force distribution and these T beams would continue into adjacent spans to collect forces in the second floor diaphragms to the damping devices. Similar drag members would be provided at the Main floor level to distribute these damping forces to the strong ground floor walls. The braced frames / reinforced concrete walls would continue through the ground story to new foundations.

2. The dome transfer and dome drum strengthening presently designed as part of the Base-Isolation Scheme by F/E is considered an appropriate scheme as part of this design, with energy dissipation devices replacing diagonal braces in the circumferential direction and within the skew truss system.

3. New shotcrete L shaped shear walls in the light court would be added to help transfer the seismic forces from the dome tower to the second floor level.

4. Additionally, it is necessary to replace damaged HCT walls where the blocks have been dislodged, and repair and strengthen (by jacketing with wire lath, plaster, and steel supports fastened to the floor and ceiling) all other HCT walls in exit ways and corridors, and where walls are excessively tall.

5. Stone facade details over exits where steel connections cannot be verified will be reanchored.

It is expected that the construction work can be performed in a manner where half of the building is vacated alternately during the 24 month construction duration.

Expected Performance: Many of the energy dissipation devices require significant local displacements or velocities to fully mobilize their energy absorption characteristics. Therefore, for earthquakes smaller than MMI VIII, the projected wall cracking damage is larger than that which would be expected to occur with shear walls. However, at ground motions at 0.30g and greater, the energy dissipation system becomes more effective than shear walls.

Even for the largest earthquakes postulated for this site, the energy dissipation system is fully functional the dissipation device support system provides additional lateral stiffness and strength in the Main floor level.

The estimated damages to walls, wall finishes, and stone work in the retrofitted building in future earthquake scenarios are given in the benefit/cost tables, Appendix 2.

3. Construction and Relocation Costs.

Table IV.3 summarizes the "hard" construction costs, associated indirect costs, a 15 percent uniform contingency allowance and a variable allowance for unforseen changes during final design and construction. Each estimated cost has been conservatively establisned to realisitcally include construction cost variabilities. Details of these cost estimates are given in Appendix 11.

C. FEDERAL DESIGN OPTION B: STRENGTHENED BUILDING DESIGN WITH EXTERIOR AND COURTYARD SHOTCRETE SHEAR WALLS, AND WITH DOME TRANSFER AND DRUM STRENGTHENING

The San Francisco City Hall building has a complete steel skeleton with unreinforced masonry infill walls. The general performance of this type of building generally has been good, even in strong ground shaking. The seismic design problems which have been identified relate to the specific layout of the building, with its large open light wells, the discontinuous exterior masonry walls (which shift from one column line to another at the first floor, the "flexible" story design, and the large weights involved at the level of the dome supported by transfer girders not connected to the exterior masonry walls.

To improve the seismic performance of San Francisco City Hall in a way which provides correction of these problems for life safety and code conformance[6], Federal Design Option "B" is based on adding overall lateral strength, providing a complete load path at those locations essential for life-safety and strengthening of the dome. The purpose of this scheme is not to provide a new lateral force resisting system but to only supplement the strength and energy absorbing capacity of the URM walls and to strengthen selected weaknesses in the lateral load path through the structure.

Significant damage reduction is achieved for more frequent small to moderate earthquake events but the purpose of the Federal funding, and thus this Federal Design Option, is not to eliminate the potential for significant damage in large events.

The Option achieves the strengthening by applying shotcrete to the inside face of all the URM infill walls, and building new concrete walls below the existing light court walls. As a result of the strengthening at the Main floor level, the strengthened building will respond with increased accelerations at the upper levels. As a result of the increased response of the dome, the drum and drum transfer levels will require a complete strengthening of its load path through the various levels of dome framing to the supporting clusters of columns.

The dome transfer and dome drum strengthening presently designed as part of the Base-Isolation Scheme by F/E is considered an appropriate scheme as part of this design, with some augmentation of the connections and member sizes. This can be justified, despite the expected higher force levels in the Federal Design, because

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[6] Current code for the retrofit of existing buildings - San Francisco Building Code, Chapter 2313.

damage control objectives are lower. With the participation of the masonry, reinforced with shotcrete, the "softening up" with the commensurate lengthening of the period and reduction in the forces documented by F/E for the Loma Prieta Earthquake, will take place in a much larger event, with the same beneficial effect. With the shotcrete reinforcement, together with the steel strengthening in the dome, the concerns over possible collapse either of the building or the dome are eliminated.

The strengthened building scheme has the following major components:

1. Reinforced shotcrete applied to the inside face of all URM walls on the perimeter of the building and on the light court walls and on the inside of the dome support columns, as shown in Figures B.1, B.2 and B.3. The shotcrete would vary in thickness from 10 to 12 inches at the base, to 6 inches above the fourth floor. In addition to supplementing the strength of the URM walls, the new shotcrete would tie the walls to the steel frame and to the floor diaphragms and thus improve the behavior of the URM portion of the walls. Interior finishes would have to be removed and replaced, but the strengthening would not have a significant impact on the size or configuration of the interior spaces. Due to the large number of windows in the light court walls, a percentage of windows would have to be closed (as is presently planned in the Forell/Elsesser design).

2. New reinforced concrete shear walls would transfer the seismic forces from the light court walls to the foundations. These would also serve to generally strengthen the lower two levels. These walls would replace some hollow clay tiles walls at the Main level and URM walls in the lower level. These walls would be 10 to 12 inches in thickness and would have up to 20% of their area open to allow for circulation.

3. New foundations for the concrete shear walls below the light courts and at the dome support columns.

4. Improved steel collectors for the new concrete walls in the diaphragms at the Main and 2nd levels. A strengthening of the diaphragms is not included since damage to the diaphragms is not a primary life-safety concern.

5. Strengthened load transfers at discontinuities at the offsets in the load path at the perimeter of the building. Where strengthened walls are not aligned, the sections of floor diaphragms require strengthening by adding a thickened shotcrete section anchored to each wall.

6. Replace damaged HCT walls where the blocks have been dislodged, and

repair and strengthen (by jacketing with wire lath, plaster, and steel supports fastened to the floor and ceiling) all other HCT walls in exit ways and corridors, and where walls are excessively tall.

7. Re-anchor stone facade details over exits where steel connections cannot be verified.

8. Strengthen steel framing supporting the dome structure utilizing a scheme with a transfer truss and strengthening similar to the F/E scheme with an increase in capacity.

The shotcrete walls serve functions in addition to adding strength. The exterior stone facade is anchored to the URM walls and thus anchored to walls that rely only on the bonding strength of the brick units. The new composite wall, reinforced shotcrete and URM, will provide an engineered wall supporting the stone elements on the facade.

The shotcrete walls will not impact the historic fabric of the building exterior and will have only minor impacts on important spaces on the interior. The layout of the scheme has been designed to completely avoid the Council Chamber, and the other rooms and spaces at the Van Ness Avenue and Polk Street Entrance bays.

In addition, impacts within the rotunda space are kept to a minimum. The strengthening of the main dome piers can be undertaken from inside the piers, and the shotcrete work on the courtyard walls will only involve the removal and replacement of the plaster on the inside face of those walls, rather than the complete demolition and rebuilding as is presently planned by the City.

The intent of the scheme is to provide seismic resistance with a base shear of approximately V=0.10W at working stress levels as described in Appendix 5. The overall strength is achieved by the combination of the shotcrete walls and the URM walls as shown in Table IV.4 which compares Option B to the story shear for V=0.10W. If the shotcrete work would result in an architectural problem because of the extra thickness, a wythe of brick masonry can be removed, but the strength of the masonry as part of the total would then be less. This level of strengthening is a compromise between retaining some of the flexibility of the original building and adding adequate strength to provide stability in the event of the severe earthquakes.

It is recognized that the original design of the dome, and that of the steel framing and diaphragms that transfer the seismic forces through the various levels of the dome to the supporting columns, was not designed for the levels of accelerations that the building may experience according to present day predictions. Strengthening is recommended in order to provide a strong and stable load path. The insertion of the supplemental damping into the dome, as described as part of Option A, could be used to improve the response of the dome by decreasing the seismic forces as part of this scheme as well. Even if it is assumed that the cost of the strengthening is not reduced substantially by including dome level energy dissipation devices, the damped strengthened scheme may have more desired behavior.

Level	Shear Capacity Option B		Story Shear V = 0.1W
	E-W	N-S	
4th Floor	13,639	12,832	9,496
3rd Floor	11,696	13,950	12,368
2nd Floor	13,007	15,296	14,576
Main Floor	20,362	20,734	16,322
Ground Floor	33,811	38,330	17,090

Table IV.4: Option B Shear Capacity

D. HISTORIC PRESERVATION CONCERNS

1. Impact of Seismic Work on Historic Fabric:

The Task Force has taken the historic importance of the structure, as well as its life-safety risks, into account in their evaluation and design option proposals. It is expected that the limited damage reduction achieved by the Federal Design Options would still be sufficient to allow relatively easy restoration of the building following most predictable earthquakes. It is highly unlikely that the building would be destroyed beyond repair in any event predicted for the site once either of the Federal Options is carried out.

The Task Force also firmly believes that either of the Federal Options, if carried to final design, could be designed in conformance with the requirements of the Secretary of the Interior's Standards for Rehabilitation, and thus would be approved under Section 106 Review of the National Preservation Act.

The issue of historic preservation has often been raised as a justification for the extra costs of the base isolation design, but it is important to note that the amount of work planned for the strengthening of the City Hall's superstructure in the City's design is extensive and involves a great deal of demolition of historic fabric. For example, (1) a visible moat must be constructed around the entire building, with seismic joints cutting across all of the entrance staircases, (2) the entire lower floor of the building must be completely gutted, (3) all of the exterior walls of the two courtyards (see photo 2A in Appendix 10) will be demolished and rebuilt, and (4) extensive demolition and reconstruction will be done in the Rotunda.[7]

Both Federal Option schemes do require some visible alterations to the interior of the building, but these alterations are not required in the most historically significant spaces. Option A will require permanent new walls in the first floor open offices on either side of both main entrances, but little construction work is required in the upper floors, except in the area of the dome. Option B will have little permanent visual effect on the interior, but will require more openning up of the interior plaster walls

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[7] It is important to note that the base-isolation technology requires a design which would ensure that the superstructure of the base-isolated building remain elastic in a design level earthquake (475 year return period), regardless of historic preservation concerns. This is required because "the interaction between the yielding superstructure and the isolation system...may result in a resonance condition which provides very undesirable structural performance" unless the superstructure is also reinforced to ensure that it remains elastic. Since assurance of elastic behavior is required by base isolation itself, the alternative of simply installing the isolators without the currently proposed extensive demolition of interior finishes and reinforcement of the superstructure is not possible. (Please see paper by James Kelly, Appendix 6, p16)

for installation of shotcrete on the interior face of the exterior walls than is necessary for Option A or the base isolation design. Most of this work is located on walls which can be easily restored. The work in the domeand dome transfer for both schemes is similar to that planned for base isolation scheme.

2 The issue of Comparative levels of Control of Damage to Historic Fabric

For the San Francisco City Hall, the City's damage control objective inherent in the base-isolation proposal increases the costs of the upgrade scheme. The code level seismic design spectra for the City Hall site are approximately equivalent to 75% of the level specified in the 1988 Uniform Building Code and the SFBC for new buildings located in seismic Zone 4 (maximum level zone). Just as is true for code conforming new buildings, it is expected that the retrofitted building by Option A and B will experience inelastic response during a design level earthquake, but that the associated deformations will not be excessive, nor the damage unacceptably severe. This performance, that is to maximize life-safety and reduce (but not necessarily eliminate) building damage, is consistent with the primary goal of the building code, and of FEMA's hazard mitigation program.

It is important to note that the elimination of building damage in major, or even moderate earthquakes, is not an objective of either the SHBC or the SFBC. The expectation is that all buildings, even historical buildings, may have to be repaired after significant earthquakes. In fact, the State Historical Building Code explicitly states that:

> *"It is not the intent to protect the property and by so doing adversely affect the historical integrity of the structure."* SHBC 8-104

The avoidance of damage at that large a level of shaking is rarely cost-effective, even for new construction. In summary, the FEMA hazard mitigation program is focused on the reduction of damage necessary to reduce life-safety risks to a reasonable level, rather than the prevention of damage per-se.

SECTION V
CONCLUSIONS AND RECOMMENDATION

REPORT SUMMARY

The Task Force recognizes that San Francisco Hall has inherent structural weaknesses that could become life-safety issues in the event of a major earthquake. Since FEMA regulations limit funds to hazard mitigation that can be shown to be "cost-effective", the role of the Task Force has been to evaluate alternative design schemes with the City's base isolation seismic retrofit scheme for different performance criteria.

It should be understood that the Task Force is not critical toward the base isolation technology or the high degree of seismic protection gained by the technology, but has been asked to evaluate alternative seismic mitigation schemes for different performance goals for comparison in order to measure the degree of appropriate FEMA funding that is waranted to meet the "cost-effective" measure. The Task Force has developed alternative designs which meet the life-safety minimum standards and the minimum seismic force levels of Section 2313 of the San Francisco Building Code.

<u>Federal Design Option A, Energy Dissipation Scheme</u>, utilizes mechanical energy dissipation devices to control building displacements and accelerations to levels which prevent building collapse in a major earthquake and controls building damage in more likely smaller earthquakes. The devices are installed in the Main floor level and in the dome and dome-to-building transfer regions. In the Main floor level the dissipation device support system is independent of the existing structural framing system to capture the relative story displacement in the Main story level for the dissipation device. The dissipation scheme successfully controls the damage of the building reasonable for all earthquake intensities.

Option A Total Project Cost (including relocation):	$83 million
Federal share = 75%	**$63 million**
Benefit/Cost Ratio:	*0.93*

<u>Federal Design Option B, Fixed Base Strengthened Building Scheme</u>, is a scheme that strengthens the building to a degree that prevents building collapse in the major

earthquake and controls damage in more likely moderate and small earthquakes. The strengthening comes from a new membrane of reinforced concrete applied to the inside face to the unreinforced masonry walls on both the perimeter and light court walls as well as strengthening of the dome, drum and drum support framing. The design option meets the FEMA Criteria outlined in Chapter IV of providing life-safety, by strengthening weaknesses in the building load path, and controlling seismic response. As a result, it will provide a degree of protection for the historical fabric by placing the strengthening such that the outward appearance of the building is unaltered and minimal changes in the interiors are required. The scheme will minimize disruption to the occupants by phasing the strengthening such that portions of the building could be under construction at one time. It is a cost effective strengthening scheme.

Option B Total Project Cost (including relocation):	$104 million
Federal share = 75%	**$78 million**
Benefit/Cost Ratio:	*0.75*

The City's Base Isolation Scheme provides seismic protection that exceeds the FEMA criteria through a combination of base isolation and building superstructure strengthening.

Base Isolation Total Project Cost (including relocation):	$180 million
Benefit-Cost Ratio:	*0.66*

TASK FORCE CONCLUSIONS

Recognizing that FEMA has concluded that the State Historical Building Code and the San Francisco Building Code are not triggered by the damage caused by the Loma Prieta Earthquake, the Panel has arrived at the following conclusions and recommendation:

1. That life-safety concerns over the future performance of the San Francisco City Hall in a large earthquake with the potential of occurring at that site can be beneficially reduced by a seismic upgrade of the building.

2. That FEMA has determined that Federal funds should be used to reduce life-safety risks in a cost-effective manner, without trying to achieve maximum property damage protection.

3. That discretionary seismic upgrade design alternatives, which are less costly than the base isolation scheme, are feasible for seismic upgrading of the San Francisco City Hall.

4. That either of the design options developed by the Panel, **Federal Design Options A and B**, would be equally or more cost effective than the City's proposal, and would satisfy FEMA program goals for total project costs, including relocation and soft costs, of approximately $83 million for Option A (**Federal Share = $63 million**) and $104 million for Option B (**Federal Share = $78 million**). The City's design has been estimated to cost approximately $180 million.

5. That the impact of either of the Federal design options on the important historical architectural features of the City Hall is different than that of the City's design, but, if carried to final design, could be kept in conformance with the Secretary of the Interior's Standards for Rehabilitation.

6. The Panel recognizes the difficulty associated with quantifying historic preservation benefits and therefore recommends that some acknowledgment of the major historical significance of San Francisco City Hall be made when comparing the different performance objectives of the different designs.

TASK FORCE RECOMMENDATION

The Panel recommends that FEMA, under the Section 406 program, which allows funds for discretionary hazard mitigation, consider making eligible the costs of a building upgrade of the San Francisco City Hall not to exceed Option B. The basis for providing some additional funds above the lower cost Option A is the Panel's concern with possible increases in costs associated with final design and construction delay of Option A,and an acknowledgment cited in item 6 above. Therefore, as stated above, the Panel would concur with a grant not to exceed the estimated cost of Option B.

APPENDIX 1

Evaluation of S.F. City Hall Main Level N/S Steel Frame Capacity

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APPENDIX 2

Benefit/Cost Analysis:
Explanation and Details

APPENDIX 3

Federal Design Option A:
Technical Details

Appendix 3

FEDERAL DESIGN OPTION A

DYNAMIC RESPONSE USING SUPPLEMENTAL ENERGY DISSIPATION DEVICES

The building is modeled as a shear building (stick model) with the building story stiffnesses, dome stiffnesses and floor weights which were determined from results presented by Forell/Elsesser in their report. The stick model properties are summarized in Table A.1. For the purposes of this analysis the energy dissipation devices are assumed to be viscous dampers with properties as given in Table A.1.

The earthquake record used for the analysis is twenty seconds of the North/South component of the El Centro (May 18, 1940) earthquake accelerogram with the acceleration intensity increased by about 1.1 to give a peak ground acceleration of 0.347 g. This adjusted accelerogram results in an acceleration response spectra which exceeds the 1989 SFBC Section 2313 elastic design forces and nearly matches the 1988 Uniform Building Code elastic design forces for a building of the same significant periods as the San Francisco City Hall, Figure A.1. Since the calculated acceleration spectra assumed a critical damping of five percent, the analytical model includes five percent stiffness proportional damping assumed for the building system, in addition to the supplemental energy dissipation devices.

In order to understand the dynamic time response of the building to the selected ground motion, it is helpful to review the dynamic modal properties of the analytical building model. The mode shape, modal period, modal damping due to the supplemental energy dissipation devices (which does not include the stiffness proportional five percent of critical damping), and the mode participation factor are summarized in Table A.2 for the first five modes. It can be seen that the first mode contains mostly office building participation, and that the second and third modes contain both dome and building responses. The fourth mode is almost exclusively dome response, but the damping in this mode is very large. It is expected that the dome response will be well controlled by this initial selection and distribution of supplemental energy dissipation devices.

The floor displacement time histories given in Figure A.2 and the story shear forces given in Figure A.3 show one major response pulse plus several cycles of lower, but significant response. It is recognized that different earthquake accelerograms will generate somewhat different responses, but this accelerogram record equals or exceeds the code design spectral responses. The maximum relative story displacements, viscous damper forces, and story shear forces occur between two and three seconds of the ground motion. These values together with the story shear capacities provided in Forell/Elsesser Table 8.1 (revised), and the ratio of elastic demand divided by shear capacity, are summarized in Table A.3. It can be seen that the maximum elastic strength demand to estimated capacity is 1.98 in the second floor level (1.51 in the Main floor level) and nearly elastic at all other levels for which data is available.

The dynamic program did not provide maximum floor accelerations as output data, so the maximum floor accelerations are estimated by dividing the maximum story shear forces by the sum of the weights of all floors above that level. This is only an approximation, but should give reasonable estimates of the elastic floor accelerations. The true floor accelerations will be lower than these elastic estimates because some of the structural members exceed their elastic limit during the earthquake. Typically the accelerations above a given level can be reduced by the amount of expected elastic strength reduction.

The weights above each story level are given in Table A.4 as calculated from the values given in Table A.1. The corresponding floor accelerations above those levels are calculated by dividing the maximum story shear from Table A.3 by the accumulated weights. The reduced accelerations for expected inelastic building response are estimated by dividing the elastic accelerations by the maximum strength ratio which occurs at any level below the floor being considered. These values are summarized in Table A.4 together with the maximum accelerations for the base isolation response provided by Forell/Elsesser in their report Figure 11.12.

From this elastic dynamic analysis it can be concluded that addition of supplemental energy dissipation devices in the tower and Main floor level can satisfactorily meet and exceed the 1989 SFBC Section 2313 requirements.

It should be noted that this conceptual study does not select the best energy dissipation device nor does it try to achieve an optimum distribution of these devices. For example, the strength ratio of 1.98 in the second story level could be reduced by placing some energy dissipation devices in that level rather than the heavy concentration of devices in the Main floor level. These refinements are beyond the scope of this conceptual study.

For the Main floor level it was decided to place energy dissipation devices at 40 locations in both the North/South and the East/West directions. Each location would carry a maximum dissipation device force of about 500 kips. The framing system was designed to allow plus and minus two inches of displacement in the Main story without the existing framing making contact with the dissipation device support system. This independent device support system in the Main floor level is supported through the ground floor level to transfer these forces to new foundations. Figures A.4 and A.5 show the device locations in the Main floor level and the device support framing system. At most locations drag members are installed in the spans adjacent to the device spans to provide relatively uniform floor diaphragm shear force transfer.

In the tower it was decided to place the energy dissipation devices in the circumferential direction at three levels similar to the braced frame system used to complement the base isolation reinforcement scheme and to place devices in combination with a reduced size skewed truss, (see Figure A.5). Although the analytical model does not accurately represent the details of the local tower behavior and consequently the best characteristics for the energy dissipation devices, the simple model does show that the tower response can be satisfactorily controlled using energy dissipation devices. The devices are conservatively priced to allow sufficient flexibility in their selection and design.

Table A.1. Building Dynamic Model Characteristics

Floor Level	Floor Weight (kips)	Story Stiffness (kips/inch)	Supplemental Viscous Damping (kip-sec / in)
Lantern floor slab	141		
dome story		1,185	
Dome floor slab	766		
drum story		1,500	100
Drum floor slab	9,013		
Octagon story		8,967	100
Octagon Deck slab	5,077		
Roof to Octagon slab		8,967	100
Pediment/Roof	24,768		
fourth story		45,625	
Fourth Floor slab	29,728		
third story		57,500	
Third Floor slab	24,892		
second story		38,000	
Second Floor slab	33,284		
main story		25,800	2,000
Main Floor slab	38,016		
ground level story		204,000	
Ground			

Table A.2. Model Dynamic Properties of the San Francisco City Hall

Mode Shape	**Mode 1**	**Mode 2**	**Mode 3**	**Mode 4**	**Mode 5**
Lantern	1.000	1.000	1.000	1.000	0.815
Dome	0.987	0.947	0.834	0.788	0.512
Drum	0.923	0.691	0.110	-0.095	-0.545
Octagon	0.813	0.340	-0.165	-0.073	1.000
Pediment	0.653	-0.096	-0.311	0.022	0.763
4th Floor	0.584	-0.159	-0.104	0.019	-0.587
3rd Floor	0.497	-0.172	0.134	-0.000	-0.704
2nd Floor	0.329	-0.142	0.373	-0.030	0.569
Main Floor	0.038	-0.017	0.054	-0.005	0.133
Period, sec	*0.977*	*0.480*	*0.271*	*0.240*	*0.180*
**Damping, %*	*12.3*	*12.6*	*25.0*	*75.8*	*7.2*
Modal Participation factor	*1.69*	*-0.98*	*0.90*	*-0.70*	*0.15*

*Note: The tabulated damping values are for the supplemental energy dissipation devices only. Five percent stiffness proportional damping is included in the dynamic response analysis.

Table A.3. Maximum Elastic Responses, Estimated Capacities, and Strength Ratio

Story Level	Maximum Relative Story Displacement (inches)	Maximum Viscous Damper Forces (kips)	Maximum Story Shear Forces (kips)	Estimated Maximum Story Shear Capacities (kips)	Story Shear Strength Ratio
Dome	0.06		75		
Drum	0.22	208	324		
Octagon	0.50	482	4,497		
Pediment	0.90	1,614	8,035		
4th Floor	0.19		8,870	14,600	0.61
3rd Floor	0.34		19,508	18,400	1.06
2 nd Floor	0.69		26,381	13,300	1.98
Main Floor	0.76	17,787	19,534	12,900	1.51
Ground	0.22		43,952	61,200	0.72

Table A.4. Estimated Elastic and Inelastic Floor Accelerations

Story Level	Sum of Weights Above (kips)	Floor Level	Elastic Floor Acceleration (g)	Inelastic Floor Acceleration (g)	Base Isolation Maximum Acceleration (g)
		Lantern	0.53	0.26	
Dome	141	Dome	0.36	0.18	0.23
Drum	907	Drum	0.45	0.22	0.22
Octagon	9,920	Octagon	0.54	0.27	0.19
Pediment	14,997	Pediment	0.22	0.11	0.18
4th Floor	39,765	4th Floor	0.28	0.14	0.18
3rd Floor	69,493	3rd Floor	0.28	0.14	0.18
2 nd Floor	94,385	2 nd Floor	0.15	0.10	0.17
Main Floor	127,669	Main Floor	0.27	0.27	0.17
Ground	165,685				

APPENDIX 4

Paper:
State-of-the-Art and State-of-the-Practice in Seismic Energy Dissipation, Robert Hanson, et.al., ATC-17-1, 1993.

APPENDIX 5

Federal Design Option B:
Technical Details

APPENDIX 5

FEDERAL DESIGN OPTION B

STRENGTHENED BUILDING DESIGN

The intent of the building strengthening scheme is to provide enough additional strength to satisfy the minimum requirements of San Francisco Building Code, Section 2313, and augment the existing strength of the masonry walls such that the combined system will provide a reasonable level of live safety. Section 2313 requires that the strengthening provide a minimum base shear as follows:

$$\mathbf{T = 0.8\ sec}$$

$$\mathbf{R_w = 6}$$

$$\mathbf{C = \frac{0.05}{\sqrt[-3]{T}} = 0.054}$$

$$\mathbf{V = \frac{8}{R_w} CW = \frac{8}{(6)}(0.054)W}$$

$$\mathbf{V = 0.072W = 12{,}305\ kips}$$

The following are the basic design assumptions used to determine the capacity of the strengthened building utilizing the capacity of the added shotcrete walls and new concrete walls below the light court walls:

Level	Shotcrete thickness	New Concrete Walls
4th Floor	6 inches	
3rd Floor	6 inches	
2nd Floor	8 inches	
Main Floor	10 inches	10 inches
Ground	12 inches	12 inches

The design assumptions recognize that the walls have numerous openings and the sophistication of the design at this time cannot model the true configuration and interaction of the openings. The capacity of the walls was based on a percentage of solid wall. The exterior walls were assumed to be 50% solid; the interior light court walls were assumed 40% solid above the second floor; and the new concrete walls below the light court walls were assumed to be 80% solid. Some openings in the light court walls will have to be closed to achieve enough solid wall. Obviously, in the further development of this design the above assumptions can be adjusted as well as the thicknesses of the walls.

The strength of the shotcrete including reinforcing is taken as 400 psi and the strength of the masonry is assumed to be one half the capacity in the Forell/Elsesser calculations. Since the design assumes that one or two wythes of brick are removed before the application of the shotcrete and the masonry and the concrete cannot share the load fully due to their different stiffnesses, the reduced masonry capacity seems justified. The capacity of the exterior stone, which may be more than the brick masonry itself, has not been included. The capacity of the building is calculated in Table B.1 and Table B.2.

The target goal was to achieve a combined concrete and masonry capacity equal to a base shear of V=0.10W. As can be seen in Table B.3 where the story shears are presented for a base shear of V=0.10W, the capacity in both directions comes close to meeting that goal. Refinements in the wall thicknesses can easily adjust the capacity to exceed the goal. In a general sense the strength of the building will be proportioned 70% to the new concrete and shotcrete walls and about 30% to the existing masonry walls.

Level	Shotcrete Shear Capacity (1)	Masonry Shear Capacity (2)	Ultimate Shear Capacity (3)=(1)+(2)	Working Shear Capacity (4)=(3)/1.7
4th Floor	12,084	11,100	23,184	13,638
3rd Floor	12,084	7,800	19,884	11,696
2nd Floor	16,112	6,000	22,112	13,007
Main Floor	29,817	4,800	34,617	20,362
Ground Floor	35,780	21,700	57,480	33,811

Table B.1 Building Capacity in the East-West Direction

Level	Shotcrete Shear Capacity (1)	Masonry Shear Capacity (2)	Ultimate Shear Capacity (3)=(1)+(2)	Working Shear Capacity (4)=(3)/1.7
4th Floor	14,515	7,300	21,815	12,832
3rd Floor	14,515	9,200	23,715	13,950
2nd Floor	19,354	6,650	26,004	15,296
Main Floor	28,799	6,450	35,249	20,734
Ground Floor	34,561	30,600	65,161	38,330

Table B.2 Building Capacity in the North South Direction

Level	**Story Shear V=0.10W**
4th Floor	9,496
3rd Floor	12,386
2nd Floor	14,576
Main Floor	16,322
Ground Floor	17,090

Table B.3 Story Shear for V=0.10W

APPENDIX 6

Paper:
State-of-the-Art and State-of-the-Practice in Base Isolation,
James M. Kelly, ATC-17-1, 1993.

APPENDIX 7

John Kariotis:
Letter to the State of California SHPO, 11/30/92on S.F. City Hall Project.

APPENDIX 8

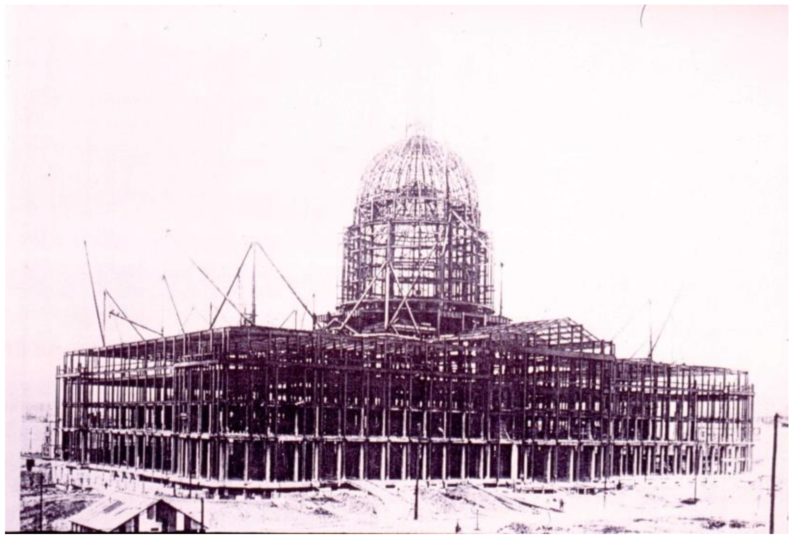
Preece Goudie & Associates, Engineers:
Review of the Forell/Elsesser Repair Study of the San Francisco City Hall, 2/16/93.

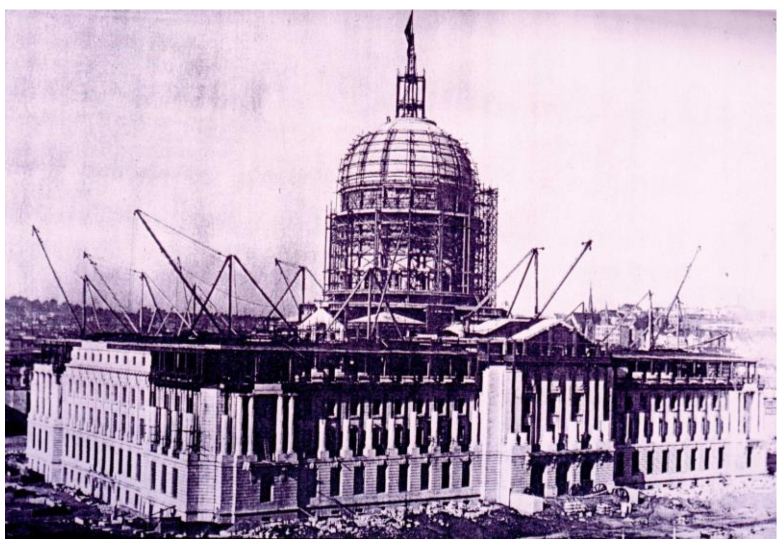
APPENDIX 9

Structural Engineers Association of Southern California:
Guidelines for Analysis of Existing Frame Structures with Concrete or Masonry Infill,
1993.

APPENDIX 10

Photographs of the San Francisco City Hall showing damage from the Loma Prieta Earthquake.



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APPENDIX 11

Detailed Cost Data on the Federal Design Options,
prepared by
P.H.Waszink, Construction Consultant, San Francisco,
May, 1994.