MIDDLEBROOK + LOUIE
Structural Engineers

PROJECT DATA SHEET

1. **Project No.** 6977 **Sub-Project No:** 00 **Date:** 1/19/05
2. **Project Name:** 301 MISSION STREET - PEER REVIEW
3. **Location:** SF **Type:** PEER REVIEW/HI-RISE OB
   Primary Code: 220 Secondary Code(s): H06
4. **Client:**
   Full Name: MILLENIUM PARTNERS
   Street & Ste.: 735 MARKET STREET, 3RD FLOOR
   City/State: SAN FRANCISCO Zip 94103
   Attn: STEPHEN M. PATTERSON Client Job No.
5. **Owner:**
   Name: MILLENIUM PARTNERS
   Street & Ste.:
   City/State: Zip
6. **Project Statistics:**
   Total Estimated Construction Cost:
   No. of Structures:
   Total Gross Area:
   No. of Stories - Above Grade:
   Below Grade:
   Estimated Completion Date:
   Bldg. Type (Steel, Concrete, etc.):
7. **Description of Services for Billing:** STRUCTURAL ENGINEERING PEER REVIEW
8. **Labor Billing:** Maximum Amount: $60,000 Estimated Amount:
   A. Normal Rates and Multipliers Yes: ☒ No: ☐
   Different Rates (indicate)
   Different Multipliers (indicate)
   B. Normal Reimbursables Yes: ☒ No: ☐
   Different Reimbursables (indicate)
9. **Fee Billing:**
   A. Fee billing schedule (Attach, if applicable.)
   B. Fee Billing by Phase: Lump Sum Fee Amount:

SD (Schematic Design)	%	or Amount
DD (Design Development)	%	or Amount
CD (Construction Documents)	%	or Amount
BN (Bidding and Negotiation)	%	or Amount
CA (Construction Administration)	%	or Amount
Other	%	or Amount

10. **Remarks:**

Project Manager: H. PANNU **Approval:** [signature]

Accounting ✓ Contract File ✓

MIDDLEBROOK + LOUIE
Structural Engineers

PROJECT DATA SHEET

1. **Project No.** 6977 **Sub-Project No:** 01 **Date:** 12/21/05
2. **Project Name:** 301 MISSION
3. **Location:** SF Type: ____________
   Primary Code: ________ Secondary Code(s): ________ ________
4. **Client:**
   Full Name: MILLENNIUM PARTNERS
   Street & Ste.: 735 MARKET STREET, 3RD FLOOR
   City/State: SAN FRANCISCO Zip 94103
   Attn: CHRIS VAUGHN-HULBERT Client Job No. ____________
5. **Owner:**
   Name ____________
   Street & Ste. ____________
   City/State ____________ Zip ________
6. **Project Statistics:**
   Total Estimated Construction Cost: ____________
   No. of Structures: ____________
   Total Gross Area: ____________
   No. of Stories - Above Grade: ____________
   Below Grade: ____________
   Estimated Completion Date: ____________
   Bldg. Type (Steel, Concrete, etc.): ____________
7. **Description of Services for Billing:** STRUCTURAL ENGINEERING REVIEW OF SHORING IMPACT ON CALTRAN BUILDING
8. **Labor Billing:** Maximum Amount: $5000 Estimated Amount: ____________
   A. Normal Rates and Multipliers Yes: ☒ No: ☐
   Different Rates (indicate) ____________
   Different Multipliers (indicate) ____________
   B. Normal Reimbursables Yes: ☒ No: ☐
   Different Reimbursables (indicate) ____________
9. **Fee Billing:**
   A. Fee billing schedule (Attach, If applicable.)
   B. Fee Billing by Phase: Lump Sum Fee Amount: ____________

Phase	%		or Amount
SD (Schematic Design)	%	________	or Amount ________
DD (Design Development)	%	________	or Amount ________
CD (Construction Documents)	%	________	or Amount ________
BN (Bidding and Negotiation)	%	________	or Amount ________
CA (Construction Administration)	%	________	or Amount ________
Other ________	%	________	or Amount ________

10. **Remarks:** ____________

Project Manager: H. PANNU **Approval:** ____________

Accounting ________ Contract File ________

6477.01

December 14, 2005

Chris Vaughn-Hulbert
Millennium Partners
735 Market Street, 3rd floor
Dan Francisco, CA 94103

RE: Structural Engineering Services Proposal for review of shoring impact on CALTRAN building
301 Mission Street, San Francisco

CHRIS, thank you for requesting our proposal for structural engineering service for the reference project.

ITEMS **FEE**

- To review the impact of shoring and about 10 to 12 feet of excavation on the CALTRAN building on the south side of property. In addition, we will fill out CALTRAN form about our findings. Our fee estimate to review and prepare CALTRAN form on T&M basis N.T.E is...................: $ 5,000

Changes in direction given to M + L which cause significant rework will be brought to your attention. Such additional compensation will be based on M + L's billing rates.

The stated fees include such things as telephone, postage and the like. We would like to be reimbursed for any printing cost, travel and subsistence (if required), express mail, express deliveries, etc.

Billing to Millennium Partners for services completed will be made at completion of work or at appropriate progress points.

CHRIS, we are pleased to provide the proposal for the above items and we look forward to continuing a long working relationship with you.

Please let us know if you have any questions.

MIDDLEBROOK + LOUIE

Hardip S. Pannu, S.E.
Principal

HSP/rhc

for MILLENNIUM PARTNERS

[signature] 12/14/05
Sign Date

Stephen Patterson, Construction Manager
Typed Name, Title

MP

MILLENNIUM PARTNERS
735 Market Street, 3rd Floor
San Francisco, CA 94103
415.537.3890 Tel
415.537.3895 Fax
SPatterson@MillenniumPtrs.com

FACSIMILE TRANSMITTAL SHEET

DATE:	January 18, 2005
TO:	Hardip Pannu
FAX NO.:	415.477.9099
FROM:	Steve Patterson
RE:	**Proposal Acceptance**
TOTAL PAGE(S)	6 including cover

☐URGENT ☐FOR YOUR INFORMATION ☐FOR REVIEW / COMMENT
☐PLEASE HANDLE ☐PER YOUR REQUEST ☒FOR YOUR FILES

NOTES/COMMENTS:

See attached.

MP

MILLENNIUM PARTNERS
735 Market Street, 3rd Floor
San Francisco, CA 94103
415.593.1100 Tel
415.537.3895 Fax

January 18, 2004

Mr. Hardip Pannu
Middlebrook + Louie
One Bush Street, Suite 250
San Francisco, CA 94104

RE: 301 Mission Street, San Francisco
Structural Engineering Peer Review Services

Dear Hardip,

I am pleased to award Middlebrook + Louie, the Structural Engineering Peer Review Services for the 301 Mission Street project, in San Francisco. The Services provided will be in accordance with your proposal dated December 17, 2004 which I have approved and attached for reference.

I very much look forward to working with yourself and team to deliver a very exciting project. I will contact you shortly to coordinate the "kick-off" meeting with the project team.

Please do not hesitate to contact me at anytime on the numbers listed above and on my attached business card.

Yours truly,

Steve Patterson
Millennium Partners

Cc: CS-011 Structural Peer Review

CS-011

MIDDLEBROOK + LOUIE
Structural Engineers

One Bush Street
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San Francisco, CA 94104
415.477.9000
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email mlbox@MplusL.com

Jason J.C. Louie, S.E.
Ronald F. Middlebrook, S.E.
Hardip S. Pannu, S.E.
Robert D. McCartney, S.E.
Jeppe Larsen, EUR ING, S.E.
Navin R. Amin, S.E.

RECEIVED
DEC 2 0 2004
MILLENNIUM PARTNERS

December 17, 2004

Stephen M. Patterson
Millennium Partners
735 Market Street, 3rd floor
San Francisco, CA 94103

RE: Structural Engineering Services Proposal
301 Mission Street - Peer Review
San Francisco, California

STEPHEN, thank you for including Middlebrook + Louie for the peer review of the referenced project. Before getting into our proposal, I'd like to briefly describe some qualified and capable peer review experience.

- 560 Mission Street Design Review: design review for the Department of Building Inspection, San Francisco City & County.
- 575 Market Street (San Francisco): peer review of a highrise office building.
- "The Century" (San Francisco): peer review of a 50-story condo tower.
- 225 W. Santa Clara (San Jose): Schematic Design of a 16-story office building. This project also entailed peer review of the final design.
- 819 Virginia Street Design Review, City of Seattle: a 34-story reinforced concrete mixed-use tower, with 9 floors for parking and the remaining floors for residential use. The gross area of the building is 350,000 square feet with the top of the building approaching 450 feet.
- 700 Olive / 1700 - 7th Street, City of Seattle: design analysis of a 23-story office tower with 7 basement levels for parking; 700,000 gross square feet. The building is of composite construction.
- 600 Van Ness (San Francisco): Peer review services for a 15-story assisted living residential tower.
- 1017 Van Ness (San Francisco): Structural value engineering and peer review services for a 14-story residential project; 250,000 sf.
- San Francisco Redevelopment Agency: M + L performed a load analysis of Marriott Hotel's underground ballroom complex to support Sony' Metreon Center.
- Cathedral Hill Apartments And Retail (San Francisco): Structural review resulting in minor modifications to original design.
- Scanticon Conference Center (Denver, Colorado): Structural review and analysis of suspended precast reinforced concrete walkways located above one another.

These and a multitude of others demonstrate that M + L has lot of peer review experience

Second, in addition to the projects listed above, M + L has a great deal of experience with large and tall projects in seismic zone 4, generally in the Bay Area. These include a half dozen of the high rise office buildings recently built in San Francisco and Oakland. On the list would be the "W" Hotel, the 26 story, 101 Second St., the 23 story 150 California, the 25 story 535 Mission Street, the 26 story One Second Street all in San Francisco, and the 22 story Elihu M. Harris State Office Building in Oakland.

Here then is our proposal:

SPatt
1/18/05

<u>SERVICES TO BE PROVIDED</u> (In general accord with SEAOC Guidelines)

A. Consideration of Design Criteria and configuration with respect to:
   1. Architectural/functional constraints.
   2. Site topography, soils, and adjacent property constraints.
   3. Environmental effects such as wind and earthquake forces.

B. Performance evaluation, including the following:
   1. Structural serviceability including deflection, lateral drift, and other movement.
   2. Vibration.
   3. Crack control.
   4. Foundation movement.
   5. Effect of deflection, lateral drift, and other movement on non-structural elements such as roof top units, etc., excluding building skin.
   6. Wind and earthquake.

C. Structural System
   1. Ability of selected structural materials and framing systems to meet performance criteria with given loads and configuration.
   2. Degree of redundancy, ductility, and compatibility, particularly in relation to lateral forces.
   3. Appropriateness of member sizes and locations.
   4. Appropriateness of foundation type and design.
   5. Compatibility of structural system and non-structural elements excluding building skin.
   6. Detailing of the structural system.
   7. Basic constructability of structural elements and connections.

D. Detailed Design
   1. Spot checking of structural calculations and/or optional independent calculations for lateral components, diaphragm design, etc.
   2. Structural design drawings and specifications for adequacy, clarity, basic constructability, and testing and inspection requirements.

M + L will discuss the findings with the Engineer of Record as the review progresses. Following the meetings and resolution of suggestions, M + L will prepare and present to the client a written report that covers all aspects of the Peer Review.

It is understood and agreed that the Peer Review is undertaken to enhance the quality of the design and to provide additional assurance regarding the performance of the completed project. Although M + L will exercise usual and customary professional care in providing this review, the responsibility for the structural design remains fully with the Engineer of Record. Accordingly, the Owner agrees to indemnify and hold M + L harmless from and against any and all claims, liabilities, demands, losses, damages, and costs (collectively, "Losses"), including but not limited to costs of defense, arising out of or in any way connected with this project excepting only those losses arising out of the sole negligence of the Peer Reviewer established by the court of law.

1/18/05

PROJECT DESCRIPTION

The project consists of a 60 story tall residential concrete building and an 11 story tall lowrise building. There is 5-story basement below the buildings. The fee is based upon Design Development structural set provided by DeSimone Consulting Engineers, printed on December 15, 2004.

FEE DATA

Basic compensation to M + L for the above described Services and Project Description for the building(s) proper shall be on Time and Material basis, not to exceed $60,000

If design input from M + L is desired during the completion of construction document phases, we would be happy to participate in that for an additional fee. The fee amount will depend on the scope of services desired.

Any significant change (increase or decrease) in the "Services To Be Provided" may cause the fees shown above to be adjusted, as agreed between the parties.

Changes in direction given M + L which cause significant rework will be brought to Millennium Partners attention. Additional compensation for any such changes will be negotiated, and authorized amounts will be billed monthly as they accrue. Such additional compensation may be based on M + L's Billing Rates, copy attached.

The stated fees include travel within the San Francisco Bay Area, telephonic, communications, postage and the like. We would like to be reimbursed for any long distance travel and subsistence required, express mail, express deliveries, etc.

Billing to Millennium Partners for services completed will be made monthly, or at appropriate progress points.

STEPHEN, please let me know if you have any questions or changes that you would like me to make to our proposal so that I can amend it accordingly.

MIDDLEBROOK + LOUIE

Hardip Singh

Hardip S. Pannu, S.E.
Principal

/hsp

S Patt
1/18/05

MIDDLEBROOK + LOUIE
Structural Engineers

2004 HOURLY BILLING RATES

Principal	$165.00 - $260.00
Project Manager / Structural Engineer	$150.00 - $185.00
Civil Engineer	$120.00 - $140.00
Design Engineer	$ 85.00 - $110.00
Construction Administrator	$110.00 - $130.00
Senior CAD Drafter	$110.00 - $155.00
CAD Drafter	$ 85.00 - $105.00
Junior CAD Drafter	$ 65.00 - $ 80.00
Administrative Staff	$ 65.00 - $105.00

Note: Hourly Billing Rates are adjusted at the beginning of each calendar year.

1/18/05

√

MIDDLEBROOK + LOUIE
Structural Engineers

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Robert D. McCartney, S.E.
Jeppe Larsen, EUR ING, S.E.
Navin R. Amin, S.E.

December 17, 2004

Stephen M. Patterson
Millennium Partners
735 Market Street, 3rd floor
San Francisco, CA 94103

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MIDDLEBROOK + LOUIE

Hardip Singh

Hardip S. Pannu, S.E.
Principal

/hsp

MIDDLEBROOK + LOUIE
Structural Engineers

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Robert D. McCartney, S.E.
Jeppe Larsen, EUR ING, S.E.
Navin R. Amin, S.E.
Carlos Y.L. Chang, S.E.
Edward X. Qi, Ph.D., S.E.
Roumen V. Mladjov, S.E.

June 26, 2006

Hanson Tom
City and County of San Francisco
1660 Mission Street, 2nd Floor
San Francisco, CA 94103

RE: 301 Mission Street – Peer Review – P/T anchor detail
San Francisco, California
M + L Job #6977

As a follow up to our final peer review letter dated June 12, 2006, we are writing this letter to state our understanding of the P/T anchors in the slab near a shear wall. Should you have any questions, don't hesitate to call us.

The slab design should include appropriate reinforcement for gravity dead and live loads and the connection to the shear wall should meet the deformation compatibility criteria per CBC section 1633.2.4. The building code provides guidelines for post-tensioned and regular cast in place slab design. In our opinion these systems can be mixed and as long as the code requirements are met for each of the system, the slab design should be acceptable. The placement of P/T anchors in the slab, outside of the shear wall effects the slab shortening due to shrinkage, but the slab to shear wall connection can be designed without the Post Tensioning cables being taken through the wall. The engineer of record has completed the design of the structure and upon verification of the design by a plan checker, the building permit should be issued.

The scope of Middlebrook + Louie's (M + L) review was to provide a professional opinions on the design based on the Building Code design provisions. The review was limited to reviewing the structural system concepts and general design approaches for compliance with requirements of the building code. It was not intended for M + L to verify the validity and/or correctness of any particular numerical values in the design calculations.

MIDDLEBROOK + LOUIE

Hardip S. Pannu, S.E.
Principal

HSP/rhc

HPANNU@MPLUSL.COM
WWW.MPLUSL.COM

(H:\ADMIN\Jobs\6977\Corres\062606 Tom.doc)

DESIMONE

NEW YORK
MIAMI
SAN FRANCISCO
NEW HAVEN
LAS VEGAS

MINUTES

FROM:	**DERRICK D ROORDA**	MEETING DATE:	**07-15-2005**
PROJECT NO.:	**4069B**	MEETING TIME:	**9:30 A.M.**
PROJECT NAME:	**301 MISSION - STRUCTURAL DESIGN SERVICES**		
MEETING LOCATION:	**SAN FRANCISCO DEPARTMENT OF BUILDING INSPECTION**		

ATTENDEES:

Gary Ho **Hanson Tom** **Y.Y. Chew**	**City and County of San Francisco - Department of Building Inspection** 1660 Mission Street, 2nd Floor, San Francisco, CA 94103	P: (415) 558-6083	F: (415) 558-6686
Derrick Roorda **Ronald Polivka** **Nicolas Rodrigues**	**DeSimone Consulting Engineers, PLLC - San Francisco** 160 Sansome Street, 16th Floor, San Francisco, CA 94104	P: (415) 490-4305	F: (415) 398-9834
Jack Moehle	**University of California, Berkeley - Earthquake Engineering Research Center** 1301 South 46th Street, Richmond, CA 94804-4698	P: (510) 231-9554	F: (510) 231-9471
Steve Patterson	**Millennium Partners - San Francisco** 735 Market Street, 3rd Floor, San Francisco, CA 94103	P: (415) 593-2500	F: (415) 537-3895
Hardip Pannu **Danil Botoshansky**	**Middlebrook + Louie Structural Engineers** One Bush Street, Suite 250, San Francisco, CA 94104	P: (415) 477-9000	F: (415) 477-9099

The following is not a comprehensive list of all comments made during the meeting, but rather is intended as a summary of key points of discussion and a list of action items to be addressed by various participants.

No.	Issue	Action
01	Introductions of all attendees were made, and their roles in the project were explained. Of special note: • J.M. has been working with DeSimone since July of 2004 and has been involved in the establishment of the design criteria and procedures. • M+L has been involved in the project since January of 2005 and are performing a peer review of the project.	N/A
02	H.T. indicated that due to the involvement of J.M. and the peer review by M+L, he is satisfied with the design and review process that is in place. He further indicated that because of this process, and the fact that the design incorporates a dual system as required by the Code, additional peer review by other outside parties will not be required by SFDBI.	N/A
03	D.R. indicated that the peer review with M+L is ongoing and presented an updated summary of all comments and responses made by DeSimone. D.R. pointed out that while several topics are still to be addressed, M+L has agreed that so long as the design of the lateral system is not changed, there are no items standing in the way of their recommending that a foundation permit be issued.	N/A

04	H.T. had the following requests of the design and review team comprised of DeSimone, J.M., and M+L: • SFDBI should be copied on all correspondence exchanged between the various parties • The design and review process should culminate with a binder containing a summary of the discussions, as well as all correspondence that was exchanged • SFDBI should be invited to attend periodic meetings with the team	DeSimone
05	H.T. indicated that the site permit, foundation permit, and superstructure permit drawing sets should include a separate drawing outlining the structural design criteria. That sheet should also contain copies of letters from both J.M. and M+L. For the site permit application, these letters should state the author's acceptance of the design criteria. For the foundation and superstructure applications, additional letters should be provided to state the author's acceptance of the design criteria, and should state that the author recommends that the permit be granted. It may be permissible for the letters to indicate that the author's recommendations are conditional upon certain issues. In such an event, SFDBI would follow up with DeSimone to insure that these conditions had been met.	DeSimone / J.M. / M+L
06	D.R. presented an overview of the structural design, including that of the foundation. Special mention was made of the capacity design approach used to limit the amount of force transferred from the outriggers to the outrigger columns through the use of link beams with diagonal reinforcing. D.R. explained that the outrigger columns have been designed to remain elastic when subjected to the full demand of all outriggers, including overstrength considerations, in addition to tributary gravity loads.	N/A
07	H.T. indicated that he liked the fact that the building includes a dual system. H.T. and G.H. inquired about the use of diagonal reinforcing in the outriggers and agreed that the approach was good for understanding the capacities of these elements. H.T. asked J.M. to review the detailing of the outriggers.	J.M.
08	D.R. discussed the steel link beams used within the core walls and explained that they had been designed per AISC requirements for "links" in EBF's.	N/A
09	D.R. indicated that, per J.M.'s suggestion and in addition to the criteria specified by the UBC, the building has been designed for the drift criteria specified by the 2003 NEHRP provisions. This approach utilizes a higher force level but allows the designer to ignore the effects of 5% mass eccentricity. H.T. requested that SFDBI be given a copy of the 2003 NEHRP provisions for review.	DeSimone
10	D.R. indicated that the tower pile cap, which includes vertical shear reinforcing, has been designed for the capacities of the lateral system elements and that this is beyond the requirements of the code. J.M. agreed that this approach is desired. H.T. asked that J.M. and M+L review the foundation design and detailing.	J.M. / M+L
11	H.T. asked about wind loads. D.R. indicated that a wind tunnel study had been performed and that the forces were much lower than those resulting from seismic loading. H.T. requested a copy of the wind tunnel report and suggested that occupant comfort be addressed. D.R. indicated that wind drifts were below typical standards for high-rise buildings and that occupant safety has been considered.	DeSimone
12	H.T. asked about detailing for PT slabs, specifically the connections to the shear core. The current drawings were reviewed. J.M. indicated that he was familiar with the concerns of SFDBI and would discuss this issue with DeSimone.	J.M. / DeSimone

MINUTES

FROM:	**DERRICK D. ROORDA**	MEETING DATE:	**02-14-2006**
PROJECT NO.:	**4069B**	MEETING TIME:	**2:00 P.M.**
PROJECT NAME:	**301 MISSION - STRUCTURAL DESIGN SERVICES**		
MEETING LOCATION:	**DESIMONE CONSULTING ENGINEERS, P.L.L.C. 160 SANSOME ST., 16TH FLOOR SAN FRANCISCO, CA 94104**		

ATTENDEES:

Gary Ho Hanson Tom Y.Y. Chew	**City and County of San Francisco - Department of Building Inspection** 1660 Mission Street, 2nd Floor, San Francisco, CA 94103	P: (415) 558-6083	F: (415) 558-6686
Derrick Roorda Ronald Polivka Nicolas Rodrigues	**DeSimone Consulting Engineers, PLLC - San Francisco** 160 Sansome Street, 16th Floor, San Francisco, CA 94104	P: (415) 490-4305	F: (415) 398-9834
Tony Sanchez-Corea III	**AR Sanchez-Corea & Associates, Inc. - San Francisco** 301 Junipero Serra Boulevard, Suite 270, San Francisco, CA 94127	P: (415) 333-8080	F: (415) 333-8990
Jack Moehle	**University of California, Berkeley - Earthquake Engineering Research Center** 1301 South 46th Street, Richmond, CA 94804-4698	P: (510) 231-9554	F: (510) 231-9471
Steve Patterson	**Millennium Partners - San Francisco** 735 Market Street, 3rd Floor, San Francisco, CA 94103	P: (415) 593-2500	F: (415) 537-3895
Hardip Pannu	**Middlebrook + Louie Structural Engineers** One Bush Street, Suite 250, San Francisco, CA 94104	P: (415) 477-9000	F: (415) 477-9099

The following is not a comprehensive list of all comments made during the meeting, but rather is intended as a summary of key points of discussion and a list of action items to be addressed by various participants.

No.	Issue	Action
01	Introductions of all attendees were made, and their roles in the project were explained.	N/A
02	H.T. explained that a lot has changed at SFDBI since we last met on July 15, 2005. There is an increased political interest in how high-rise buildings are designed and reviewed. More peer review meetings need to occur with the city's participation. D.R. explained that there have been no peer review meetings since our July 15 meeting, and that SFDBI will be invited to attend all future meetings.	N/A
03	S.P. indicated that the shoring work for the tower is complete, the soil mix wall is installed, and pile driving is to start the week of February 20.	

04	H.T. asked both J.M. and H.P. if they had reviewed the foundation permit package in detail. Both J.M. and H.P. indicated that they had reviewed the foundation package calculation package, plans and details, and that they were satisfied that the foundation meets or exceeds the requirements set forth in the building code.	N/A
05	D.R. indicated that because the design of the project complies with the code, it is permissible to take the results from a response spectrum analysis, combine them with gravity loads, and design the foundation for those forces. However, at the suggestion of J.M., the foundation has been designed for the capacities of the lateral system elements. This is beyond code, and insures extra capacity in the foundation.	
06	H.P. indicated that the foundation was designed for the capacities of the building and so the foundation design was designed for more than what was required by code. In his opinion, this design philosophy was more than adequate.	N/A
07	H.T. asked H.P. if Middlebrook had looked into the assumptions of the analysis model. H.P. indicated that they looked into the analysis model created by DeSimone and understands the assumptions made. He further explained that in order to perform an independent check of DeSimone's design forces, M&L created their own analysis model. After comparing the two models, H.P. was satisfied that DeSimone's model was comparable.	N/A
08	H.T. asked J.M. if he had looked into the assumptions of the analysis model. J.M. explained that he had been advising during the inception of DeSimone's analysis model. He indicated that it is his recollection that many model assumptions had been changed and updated at his request. He also explained that the 301 model is Linear Response Spectrum Analysis, and that this type of model is different than the Non-Linear Time History Analysis models being used on other projects. The model used for 301 does not require as much scrutiny, and the models assumptions are mainly dictated by code.	N/A
09	H.T. asked the peer reviewers if they required more time to perform an adequate check of the design. Both H.P. and J.M. indicated that more time was not necessary, the foundation design meets or exceeds the codes requirements, and that they have provided letters indicating their positions on this matter.	N/A
10	G.H. asked about effects of Transbay terminal on the project. S.P. and D.R. explained the status of negotiations with the Transbay joint Power Authority. H.T. indicated that it is not the responsibility of the design team or the peer reviewers to review this information.	N/A
11	H.T. asked about how the foundation was modeled and specifically asked about pier springs in model, and interaction with the mid-rise building. D.R and N.R. explained that the buildings are completely separate. D.R. explained that Treadwell & Rollo were familiar with DeSimone's design procedures, have reviewed the design, and their letter is included on the foundation permit drawings. D.R. explained that T&R consider the pile cap to be supported almost continuously, much like a mat foundation, and that T&R recommended it be analyzed as a mat with varying stiffness under different areas, according to the expected displacements. D.R. and N.R. explained that an area spring matching the overall foundation stiffness was used in the ETABS analysis for the superstructure. H.T. asked J.M. if this was done properly, and J.M. responded that he thought the assumption was appropriate. H.P. indicated that they M+L made their own ETABS model to check this assumption and agreed that it is appropriate.	N/A

12	H.T. asked both J.M. and H.P. if they had checked calculations specifics, including rebar quantities. H.P. indicated that that level of review was beyond a peer review level and therefore outside their current scope of services. H.P. further indicated that if the city was interested, his firm could provide a plan check level of review under an additional scope of services. J.M. indicated that he too could provide a plan check level of review, but this more detailed level of review is also outside his current scope of services. J.M. indicated that this level of review is beyond what has been asked by SFDBI of peer reviewers for other high-rise projects in the city.	
13	Y.Y.C. suggested that DeSimone meet with G.H. and explain the building design procedures for the superstructure in more detail. H.P and D.R. agreed that this may help speed the SFDBI review process.	N/A
14	H.T. requested that H.P. and J.M. bring the drawings they reviewed for the foundation permit submittal to SFDBI to compare with the official permit drawing set. A meeting time was set for 2/16/05 at 2pm at D.B.I. D.R. indicated that he would attend the meeting also. H.T. indicated that once this was complete the foundation permit would be issued.	D.R., J.M., H.P., & G.H.
15	H.T. requested that DeSimone meet with SFDBI to discuss criteria and procedures used to design the superstructure. Meeting was set for 2/22/06 at 2pm at SFDBI	D.R. & G.H.
16	D.R. requested that a superstructure peer review meeting be scheduled. Meeting was set for 3/9/06 at 2pm at DeSimone's office.	All

June 12, 2006

Steve Patterson
735 Market Street, 3rd Floor
San Francisco, CA 94103

RE: 301 Mission Street – Peer review, Final.
San Francisco, California
M + L Job #6977

We have completed the peer review of the super structure design prepared by DeSimone Consulting Engineers for the 301 Mission Street project dated May 26, 2006. Our peer review included only the review of 58-story tower. The engineer of record's decision was to design this building to conform to the 2001 San Francisco Building Code and our peer review followed the same approach.

Our entire peer review comments and responses are included in the two binders (Peer Review 1 and 2, dated May 31st, 2006) compiled by DeSimone Consulting Engineers.

- Our peer review included key details and major components of the building system, such as design of shear walls and shear links, design of moment frames, column shortening etc. There were two comments (comment 11 and 20) where the engineer of record took exception to our suggestions. Based on our review of the project, it is our opinion that the design of the tower follows the general principals of engineering design and after the plan check review by the City a permit can be issued for construction.

- We were not asked to review the effects of the Transbay Terminal project on this project.

The engineer of record has completed the design of the structure. It is our understanding that the scope of Middlebrook + Louie's (M + L) review was to provide our professional opinions on the design based on the Building Code design provisions. We also understand that M + L's review is limited to reviewing the structural system concepts and general design approaches for compliance with requirements of the building code. It is not intended for M + L to verify the validity and/or correctness of any particular numerical values in the design calculations.

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June 26, 2006

Hanson Tom
City and County of San Francisco
1660 Mission Street, 2nd Floor
San Francisco, CA 94103

RE: 301 Mission Street – Peer Review – P/T anchor detail
San Francisco, California
M + L Job #6977

As a follow up to our final peer review letter dated June 12, 2006, we are writing this letter to state our understanding of the P/T anchors in the slab near a shear wall. Should you have any questions, don't hesitate to call us.

The slab design should include appropriate reinforcement for gravity dead and live loads and the connection to the shear wall should meet the deformation compatibility criteria per CBC section 1633.2.4. The building code provides guidelines for post-tensioned and regular cast in place slab design. In our opinion these systems can be mixed and as long as the code requirements are met for each of the system, the slab design should be acceptable. The placement of P/T anchors in the slab, outside of the shear wall effects the slab shortening due to shrinkage, but the slab to shear wall connection can be designed without the Post Tensioning cables being taken through the wall. The engineer of record has completed the design of the structure and upon verification of the design by a plan checker, the building permit should be issued.

The scope of Middlebrook + Louie's (M + L) review was to provide a professional opinions on the design based on the Building Code design provisions. The review was limited to reviewing the structural system concepts and general design approaches for compliance with requirements of the building code. It was not intended for M + L to verify the validity and/or correctness of any particular numerical values in the design calculations.

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Robert D. McCartney, S.E.
Jeppe Larsen, EUR ING, S.E.
Navin R. Amin, S.E.

August 30, 2005

Hanson Tom
City and County of San Francisco
1660 Mission Street, 2nd Floor
San Francisco, CA 94103

RE: 301 Mission Street – Foundation Permit Only
San Francisco, California
M + L Job #6977

We have completed the peer review of the foundation system prepared by DeSimone Consulting Engineers for the 301 Mission Street project dated May 24, 2005 for the Foundation Permit Submittal Only with following assumptions and exceptions:

The design of the superstructure has not been completed at this time. Our understanding from meetings with DeSimone is that the superstructure's lateral system will be designed to comply with the following:

- The outriggers connecting to the central shear core of the tower contains links connecting to the Special Moment Resisting Frame columns. These links will be designed to remain elastic under the code-prescribed Gravity, Wind and Seismic load combinations; including loads caused by column shortening effects in tall buildings.
- The Special Moment Resisting Frame Columns will be designed to remain elastic under gravity plus loads caused by the yielding of outrigger link. In order to ensure this behavior, the capacities of the outrigger links will be calculated and increased by an over-strength factor. The resulting forces were used as the seismic loads.
- The pile cap under the tower is designed to remain elastic when subjected to the capacities of the Special Moment Resisting Frame/outrigger columns, as well as the expected maximum moment at the base of the shear wall core.
- We were not asked to review the effects of the Transbay Terminal project on this project.

The Structural Peer Review is ongoing at this time for the superstructure portion. It is our understanding that the scope of Middlebrook + Louie's (M + L) review is to provide our professional opinions on the design based on the Building Code design provisions. We also understand that M + L's review is limited to reviewing the structural system concepts and general design approaches for compliance with requirements of the building code. It is not intended for M + L to verify the validity and/or correctness of any particular numerical values in the design calculations.

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August 30, 2005
Revised Jan 24th, 2006

Hanson Tom
City and County of San Francisco
1660 Mission Street, 2nd Floor
San Francisco, CA 94103

RE: 301 Mission Street – Foundation Permit Only
San Francisco, California
M + L Job #6977

We have completed the peer review of the foundation system prepared by DeSimone Consulting Engineers for the 301 Mission Street project dated May 24, 2005 for the Foundation Permit Submittal Only including all the structural drawings listed on sheet S0.01 with following assumptions and exceptions:

The design of the superstructure has not been completed at this time. Our understanding from meetings with DeSimone is that the superstructure's lateral system will be designed to comply with the following:

- The outriggers connecting to the central shear core of the tower contains links connecting to the Special Moment Resisting Frame columns. These links will be designed to remain elastic under the code-prescribed Gravity, Wind and Seismic load combinations; including loads caused by column shortening effects in tall buildings.

- The Special Moment Resisting Frame Columns will be designed to remain elastic under gravity plus loads caused by the yielding of outrigger link. In order to ensure this behavior, the capacities of the outrigger links will be calculated and increased by an over-strength factor. The resulting forces were used as the seismic loads.

- The pile cap under the tower is designed to remain elastic when subjected to the capacities of the Special Moment Resisting Frame/outrigger columns, as well as the expected maximum moment at the base of the shear wall core.

- We were not asked to review the effects of the Transbay Terminal project on this project.

The Structural Peer Review is ongoing at this time for the superstructure portion. It is our understanding that the scope of Middlebrook + Louie's (M + L) review is to provide our professional opinions on the design based on the Building Code design provisions. We also understand that M + L's review is limited to reviewing the structural system concepts and general design approaches for compliance with requirements of the building code. It is not intended for M + L to verify the validity and/or correctness of any particular numerical values in the design calculations.

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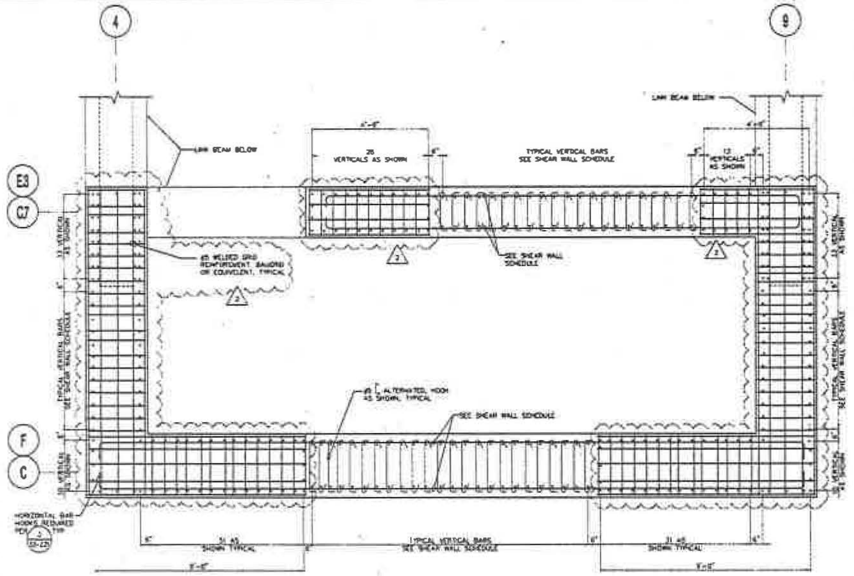

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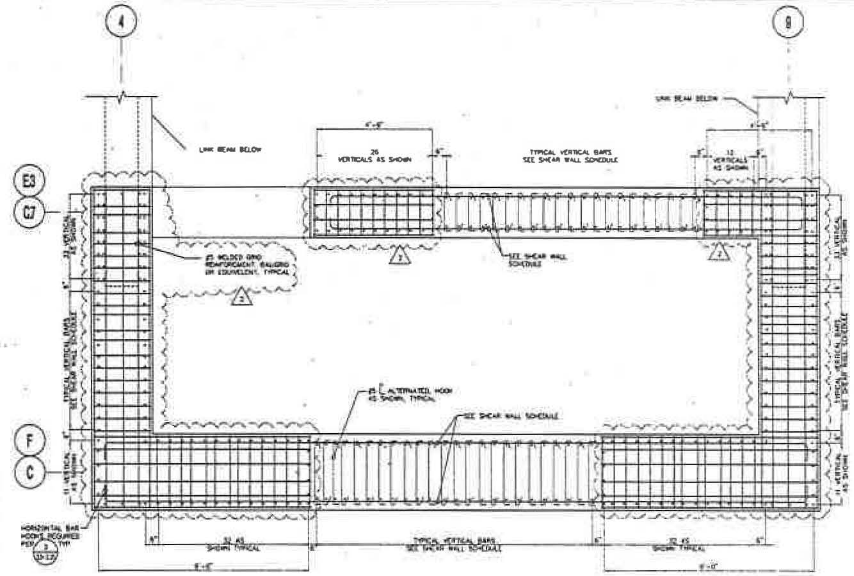

1 **SHEAR WALL DETAIL**
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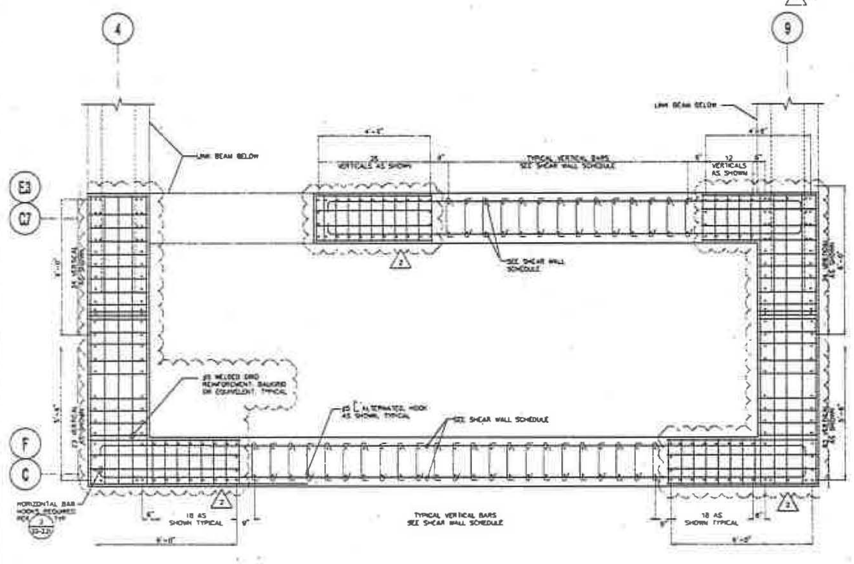

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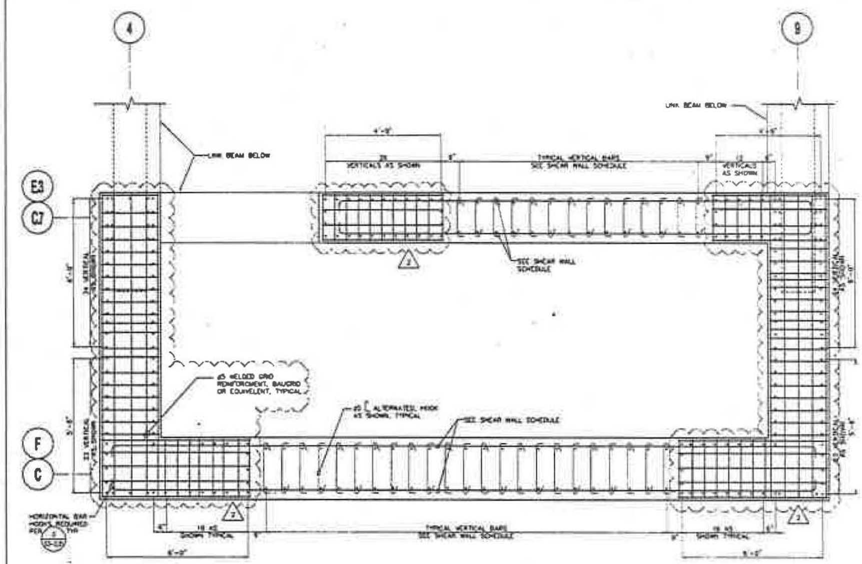

3 **SHEAR WALL DETAIL**
SCALE: 1/2"=1'-0"

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REV	DESCRIPTION	DATE
	FOUNDATION PERMIT	06/24/2005
1	ADDENDUM 2 STRUCTURE	11/18/2005
	ADDENDUM 2 REVISIONS	03/06/2006
2	ADD2-REV3 BAUGRID	09/25/2006

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DERRICK D. ROORDA
NO. S 4807
STRUCTURAL
STATE OF CALIFORNIA

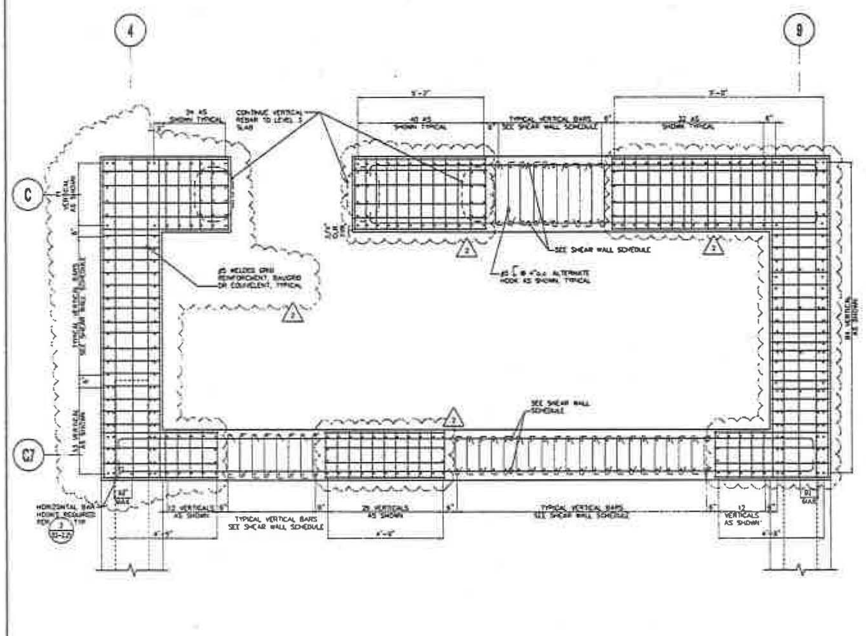
SCALE: AS NOTED
DRAWN BY: ELS
PROJECT #: 4069

PROJECT TRUE

VERTICAL SYSTEMS

SHEAR WALL DETAILS

S3-2.53



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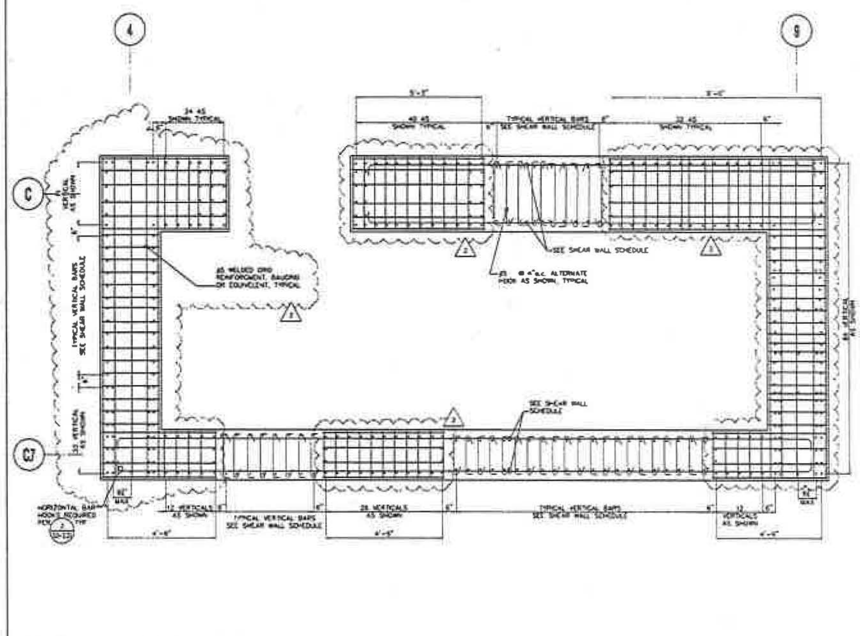

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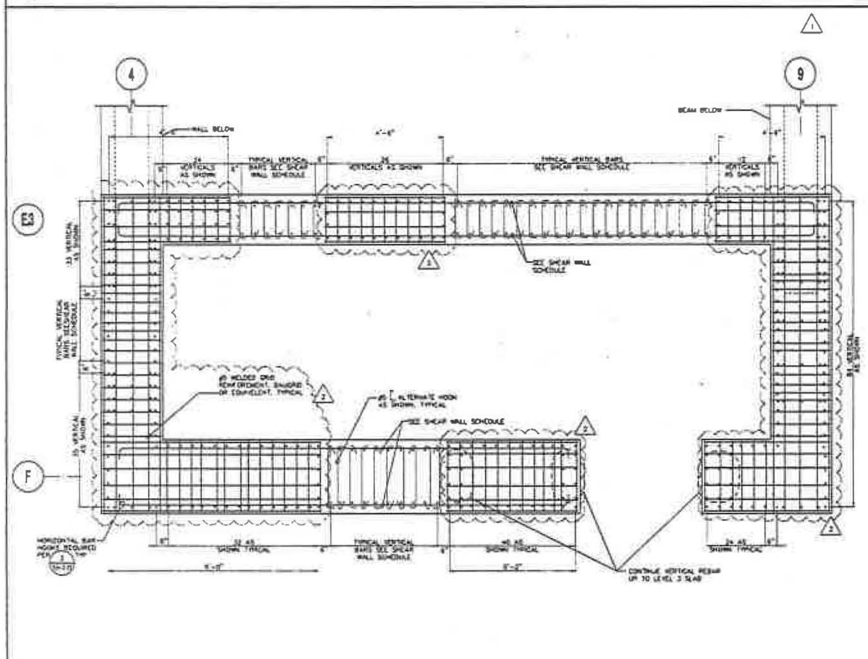

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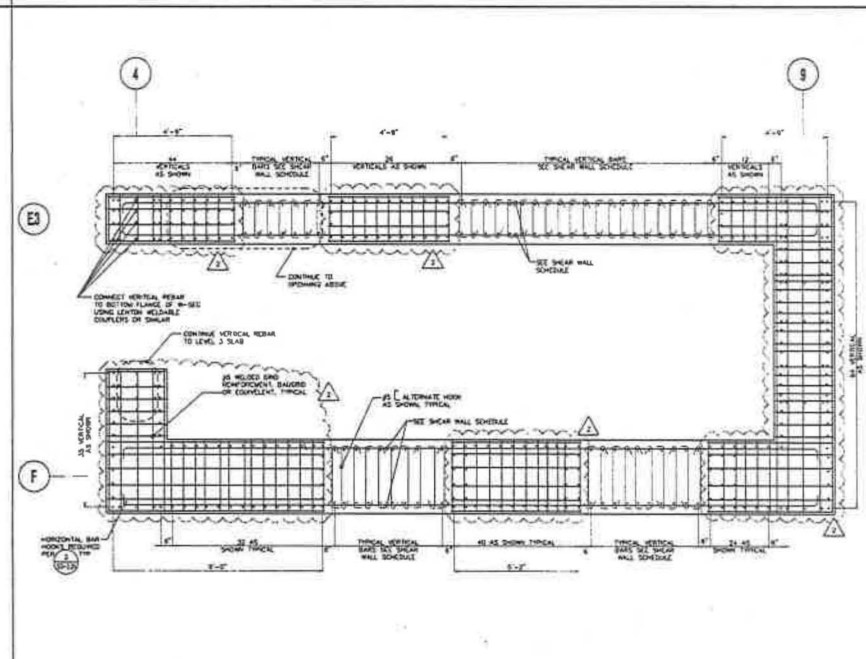

3 SHEAR WALL DETAIL - LEVEL B1
SCALE: 1/2"=1'-0"

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REV.	DESCRIPTION	DATE
	FOUNDATION PERMIT	05/24/2005
1	ADDENDUM 2 STRUCTURE	11/18/2005
	ADDENDUM 2 REVISIONS	03/06/2006
2	ADD2-REV3 BAUGRID	09/25/2006

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SCALE AS NOTED
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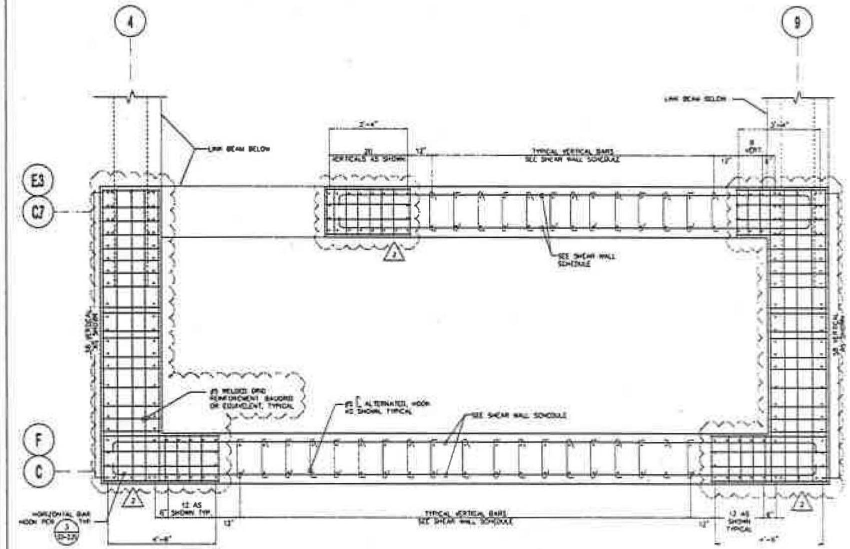
PROJECT # 4060

PROJECT TRUE

VERTICAL SYSTEMS

SHEAR WALL DETAILS

S3-2.51



1 SHEAR WALL DETAIL

SCALE: 1/2"=1'-0"

2 NOT USED

SCALE

3 NOT USED

SCALE

4 NOT USED

SCALE

301 MISSION STREET

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REV.	DESCRIPTION	DATE
	FOUNDATION PERMIT	06/24/2005
1	ADDENDUM 2 STRUCTURE	11/18/2005
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2	ADD2-REV3 BAUGRID	09/25/2006

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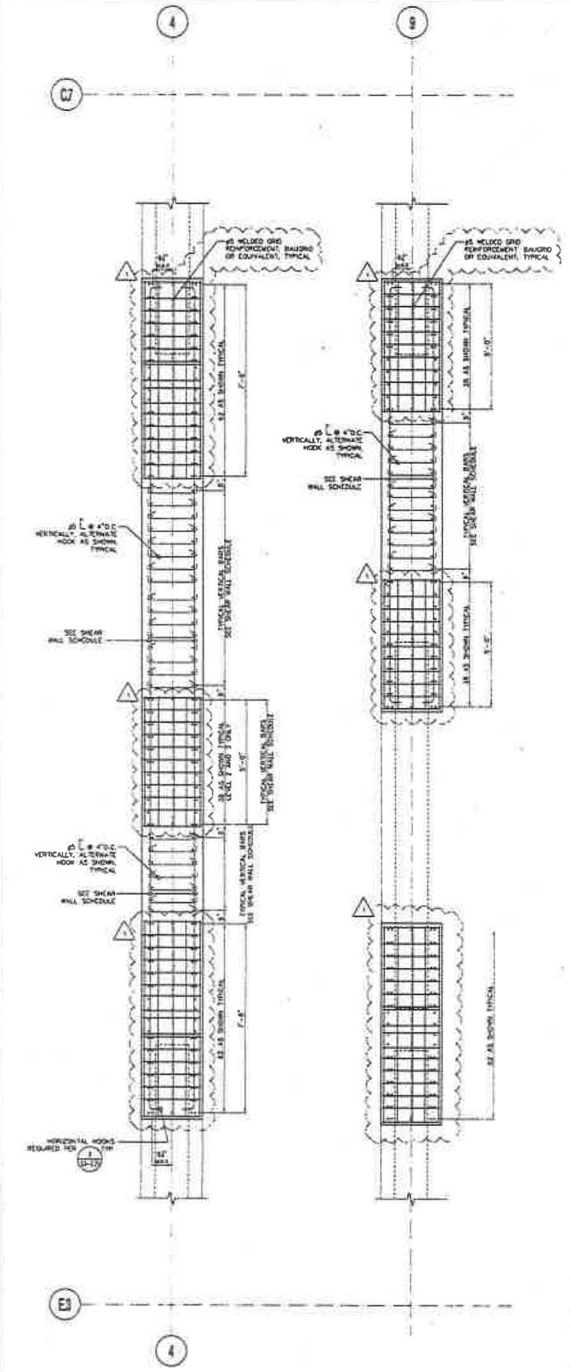
SCALE: AS NOTED
DRAWN BY: ELS
PROJECT #: 4069

VERTICAL SYSTEMS

SHEAR WALL DETAILS

S3-2.54



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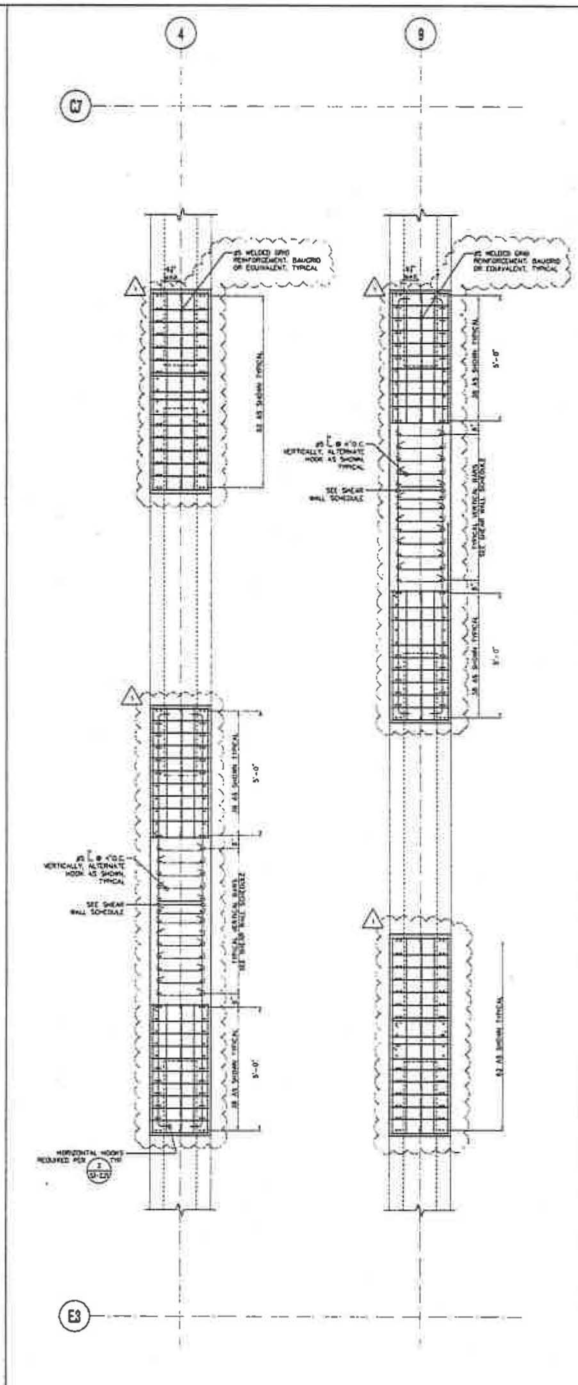

2 SHEAR WALL DETAIL - LEVEL 01
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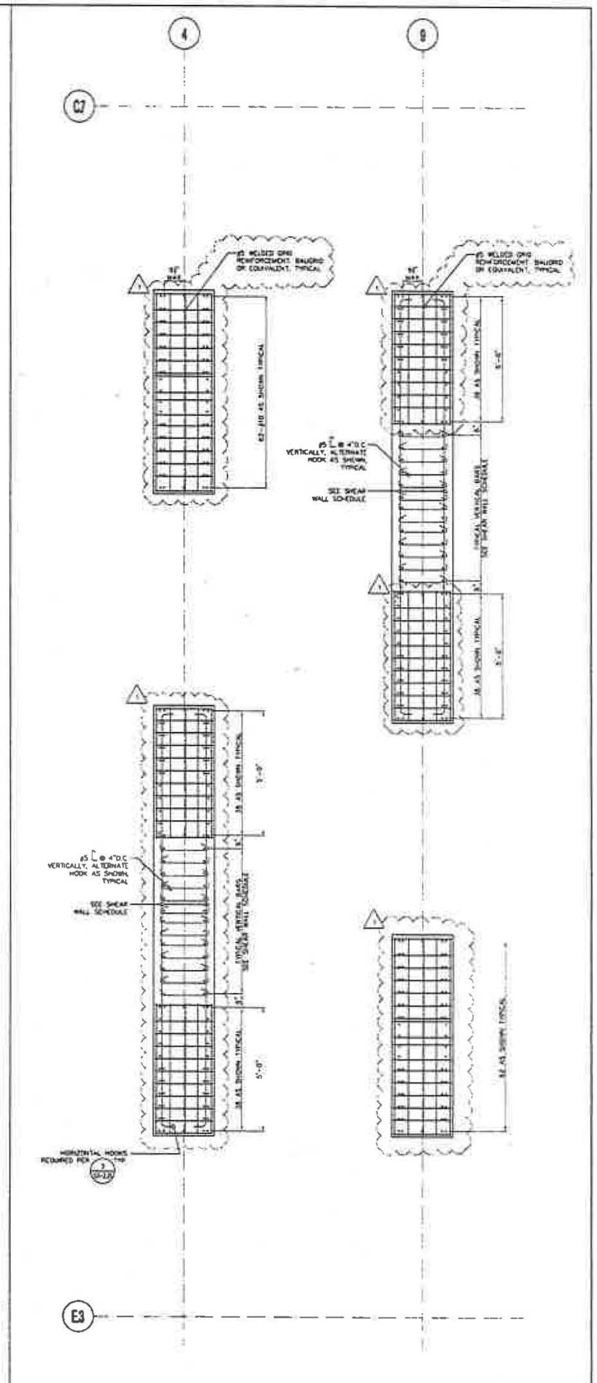

1 SHEAR WALL DETAIL - LEVEL B1
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REV.	DESCRIPTION	DATE
	FOUNDATION PERMIT	05/24/2005
	ADDENDUM 2 STRUCTURE	11/16/2005
	ADDENDUM 2 REVISIONS	03/06/2006
1	ADD2-REV3 BAUGRID	09/25/2006

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EXP. 06-30-07
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STATE OF CALIFORNIA

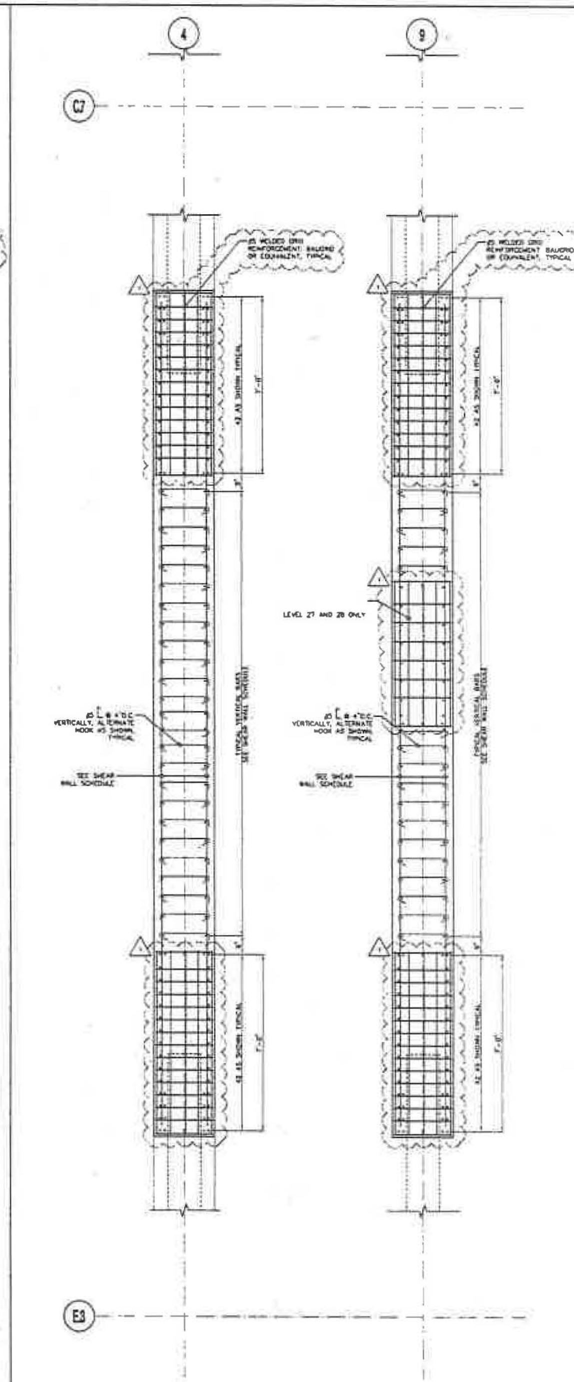
SCALE: AS NOTED
DRAWN BY: ELS
PROJECT #:

N
PROJECT
TRUE

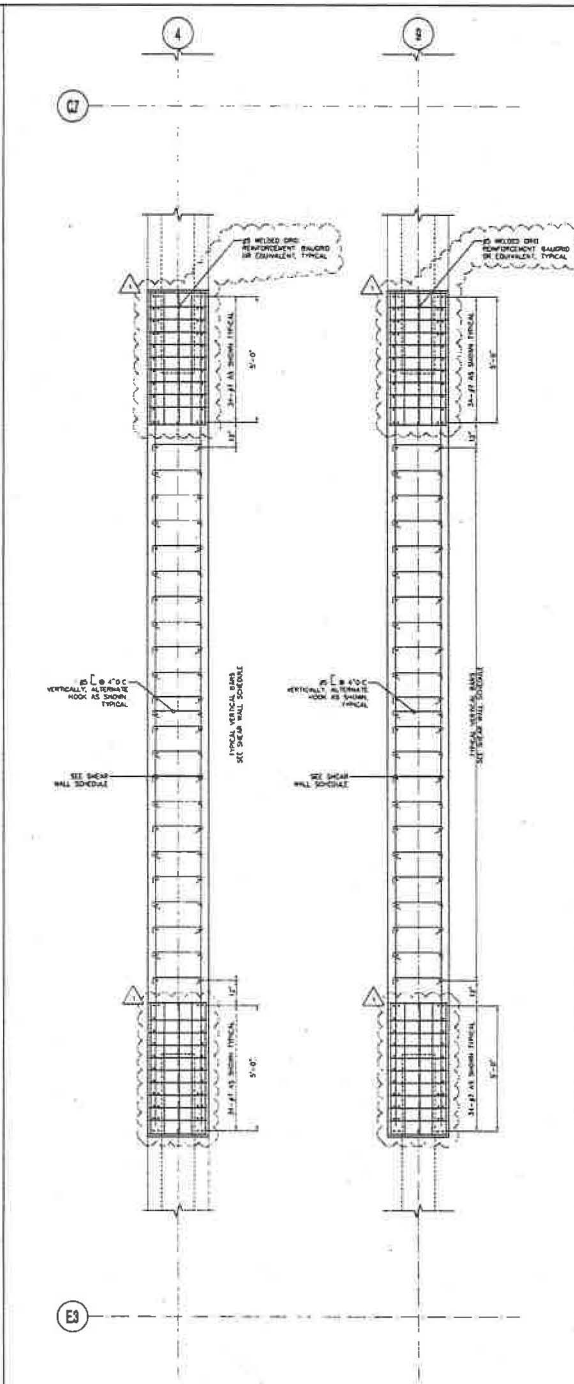
VERTICAL SYSTEMS

SHEAR WALL DETAILS

S3-2.55



1 SHEAR WALL DETAIL
SCALE: 1/2"=1'-0"



2 SHEAR WALL DETAIL
SCALE: 1/2"=1'-0"

3 NOT USED
SCALE

301 MISSION STREET

OWNER:
MISSION STREET DEVELOPMENT PARTNERS LLC 735 MARKET STREET 3RD FLOOR SAN FRANCISCO, CA 94103

ARCHITECT:
HANDEL ARCHITECTS, LLP
735 MARKET ST., 2ND FLR. SAN FRANCISCO, CA 94103
1865 BROADWAY, 4TH FLOOR NEW YORK, NEW YORK, 10023

STRUCTURAL ENGINEER:
DESIMONE CONSULTING ENGINEERS
160 SANSOME STREET, 16TH FLOOR SAN FRANCISCO, CA 94104-3722

MECHANICAL/ELECTRICAL/PLUMBING ENGINEER:
FLACK & KURTZ INC.
405 HOWARD STREET, STE. 500 SAN FRANCISCO, CA 94105
475 5TH AVE NEW YORK, NEW YORK

FIRE PROTECTION ENGINEER:
HUGHES ASSOCIATES
2551 SAN RAMON VALLEY BLVD. STE 205 SAN RAMON, CA

CODE CONSULTANT:
ARC & ASSOCIATES
301 JUNIPERO SERRA BLVD. STE 270 SAN FRANCISCO, CA

REV.	DESCRIPTION	DATE
	ADDENDUM 2 STRUCTURE	11/18/2005
	ADDENDUM 2 REVISIONS	03/08/2006
1	ADD2-REV3 BAUGRID	08/25/2006

DESIMONE
160 SANSOME STREET 16TH FLOOR
SAN FRANCISCO, CA 94104-3722
T 415.398.5740 F 415.398.9834

STAMP:

REGISTERED PROFESSIONAL ENGINEER DERRICK D. ROORDA NO. S 4607 EXP. STRUCTURAL STATE OF CALIFORNIA

SCALE: AS NOTED
DRAWN BY: ELS
PROJECT #:

PROJECT TRUE

VERTICAL SYSTEMS
SHEAR WALL DETAILS

S3-2.56

TOWER MOMENT FRAME BEAM SCHEDULE

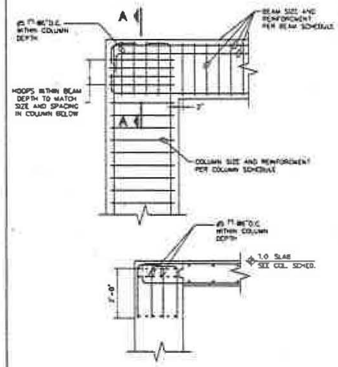
MARK	LEVEL	SIZE (WIDTH x DEPTH)	ORIENTATION SEE 13/-	TOP BARS CONT	TOP BARS ADDED	BOTTOM BARS CONT	BOTTOM BARS ADDED	HOOPS SIZE	HOOPS END SPAN LEGS	HOOPS END SPAN SPACING	HOOPS MID SPAN LEGS	HOOPS MID SPAN SPACING
B1	48-60	24 x 28	UP	8-#10	2-#10	6-#10	2-#10	#4	6	6	6	12
	41-47	24 x 28	UP	6-#10	4-#10	5-#10	3-#10	#4	6	6	6	12
	31-40	24 x 30	UP	8-#10	5-#10	5-#10	4-#10	#4	6	6	6	10
	21-30	24 x 30	UP	8-#10	5-#10	8-#10	6-#10	#4	6	5	6	8
	15-20	24 x 34	UP/DOWN	6-#10	8-#10	6-#10	6-#10	#4	6	4.5	6	7
	4-14	24 x 38	UP/DOWN	8-#10	8-#10	8-#10	8-#10	#4	6	4	6	8
	3	24 x 38	DOWN	6-#10	8-#10	6-#10	8-#10	#4	6	4	6	6
	2	24 x 38	UP/DOWN	6-#10	8-#10	6-#10	6-#10	#4	6	4	6	6
	1	24 x 38	DOWN	6-#10	6-#10	6-#10	8-#10	#4	6	4	6	6
B2	58	24 x 31	UP	6-#10	2-#10	6-#10	N/A	#4	6	8	6	12
	41-57	24 x 28	UP	8-#10	2-#10	6-#10	N/A	#4	6	6	6	12
	31-40	24 x 30	UP	6-#10	6-#10	6-#10	4-#10	#5	6	6	6	10
	21-30	24 x 30	UP	6-#10	8-#10	6-#10	6-#10	#5	6	6	6	8
	15-20	24 x 34	UP/DOWN	6-#10	8-#10	6-#10	8-#10	#5	6	5	6	8
	4-14	24 x 38	UP/DOWN	6-#10	8-#10	6-#10	6-#10	#5	6	5	6	7
	3	24 x 38	DOWN	6-#10	8-#10	6-#10	6-#10	#5	6	5	6	7
	2	24 x 38	UP/DOWN	6-#10	8-#10	6-#10	6-#10	#5	6	5	6	7
	1	24 x 38	DOWN	6-#10	8-#10	6-#10	8-#10	#5	6	5	6	7
B3	58	24 x 31	UP	6-#9	2-#9	6-#9	N/A	#4	6	8	6	12
	48-57	24 x 28	UP	6-#9	2-#9	6-#9	N/A	#4	6	6	6	12
	41-47	24 x 28	UP	6-#9	4-#9	6-#9	2-#9	#4	6	5	6	9
	21-40	24 x 30	UP	6-#10	6-#10	6-#10	4-#10	#5	6	5	6	7
	15-20	24 x 34	UP/DOWN	6-#10	8-#10	6-#10	4-#10	#5	6	3	6	7
	4-14	24 x 38	UP/DOWN	8-#10	8-#10	6-#10	8-#10	#5	6	4.5	6	6
	3	24 x 38	DOWN	6-#10	5-#10	6-#10	5-#10	#5	6	4.5	6	6
	2	24 x 38	SEE PLAN	6-#10	6-#10	6-#10	6-#10	#5	6	4.5	6	6
	1	24 x 38	DOWN	6-#10	6-#10	6-#10	6-#10	#5	6	4.5	6	6
B4	41-58	24 x 28	UP	10-#9	N/A	6-#9	N/A	#4	4	6	4	6
	21-40	24 x 30	UP	10-#9	N/A	6-#9	N/A	#4	4	6	4	6
	15-20	24 x 34	UP/DOWN	10-#9	N/A	6-#9	N/A	#4	4	6	4	6
	4-14	24 x 38	UP/DOWN	10-#9	N/A	6-#9	N/A	#4	4	6	4	6
	3	24 x 38	DOWN	10-#9	N/A	6-#9	N/A	#4	4	6	4	6
	2	24 x 38	SEE PLAN	10-#9	N/A	6-#9	N/A	#4	4	6	4	6
	1	24 x 38	DOWN	10-#9	N/A	6-#9	N/A	#4	4	6	4	6
B5	48-61	24 x 28	UP	6-#9	2-#9	6-#9	N/A	#4	6	6	6	12
	41-47	24 x 28	UP	6-#9	4-#9	6-#9	2-#9	#4	6	6	6	12
	21-40	24 x 30	UP	6-#9	6-#9	6-#9	4-#9	#4	6	6	6	10
	15-20	24 x 34	UP/DOWN	6-#9	6-#9	6-#9	6-#9	#4	6	6	6	10
	4-14	24 x 38	UP/DOWN	6-#10	5-#10	6-#10	6-#10	#4	6	4.5	6	6
	3	24 x 38	DOWN	6-#10	6-#10	6-#10	6-#10	#4	6	4.5	6	6
	2	24 x 38	SEE PLAN	6-#10	6-#10	6-#10	6-#10	#4	6	4.5	6	6
	1	24 x 38	DOWN	6-#10	6-#10	6-#10	6-#10	#4	6	4.5	6	6
B6	3	36 x 42	DOWN	12-#10	6-#10	12-#10	6-#10	#5	6	4	6	6
	2	36 x 66	SEE NOTE 4	16-#10	6-#10	16-#10	6-#10	#5	6	4	6	6
	1	36 x 66	SEE NOTE 4	12-#10	6-#10	12-#10	6-#10	#5	6	4	6	6

NOTES:

1. fy = 75 ksi (#9 OR LARGER)
2. fy = 60 ksi (#8 OR SMALLER)
3. f'c = MATCH COLUMN STRENGTH PER COLUMN SCHEDULE
4. SEE SHEET S3-2.72

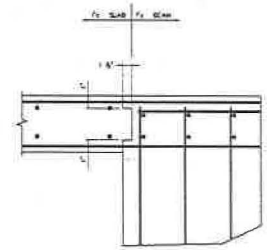
15 TOWER MOMENT FRAME BEAM SCHEDULE

3 NOT USED
SCALE:

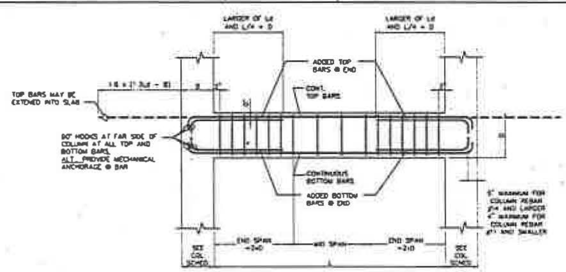


SECTION A-A

2 TYPICAL BEAM-COLUMN KNEE JOINT DETAIL
SCALE: 1/2"=1'-0"



1 BEAM - SLAB KEY DETAIL
SCALE: N.T.S.



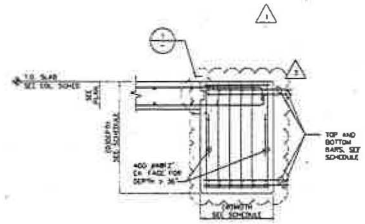
DEVELOPMENT LENGTHS FOR MOMENT FRAME BEAM BARS

fy	BAR SIZE	f'c = 7,000 psi	Ld (in) TOP BARS	Ld (in) OTHER BARS	f'c = 8,000 psi	Ld (in) TOP BARS	Ld (in) OTHER BARS	f'c = 10,000 psi	Ld (in) TOP BARS	Ld (in) OTHER BARS
60 ksi	#7		34	34		33	32		30	28
	#8		38	39		35	36		33	32
75 ksi	#9		39	54		38	51		33	45
	#10		42	60		40	56		36	50
	#11		47	66		44	62		40	56

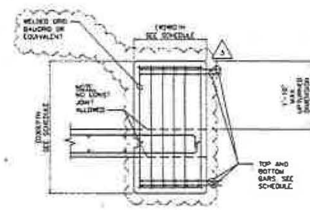
NOTES:

1. Ld = TENSION DEVELOPMENT LENGTH PER SCHEDULE THIS DETAIL
2. Ldh = TENSION DEVELOPMENT LENGTH FOR STANDARD HOOK PER SCHEDULE THIS DETAIL
3. SEE TOWER MOMENT FRAME BEAM SCHEDULE FOR BEAM SIZE AND REINFORCING
4. LONGITUDINAL BARS MUST PASS BETWEEN COLUMN VERTICAL BARS UNLESS SPECIFICALLY SHOWN OR NOTED OTHERWISE.

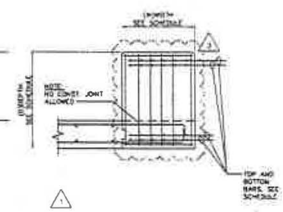
7 TYPICAL CONCRETE MOMENT FRAME BEAM DETAIL
SCALE: N.T.S.



DOWNTURNED BEAM



UP/DOWN BEAM



UPTURNED BEAM

NOTES:

1. IF DEPTH <= 31" BEAM IS UPTURNED, U.N.O. IN SCHEDULE
2. IF DEPTH > 31" BEAM IS UP/DOWN, U.N.O. IN SCHEDULE
3. MAXIMUM OF 5 LONGITUDINAL BARS PER LAYER. PROVIDE 1" CLEAR DISTANCE BETWEEN LAYERS.

13 TYPICAL TOWER MOMENT FRAME DETAILS
SCALE: 3/4"=1'-0"

301 MISSION STREET

OWNER:
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FIRE PROTECTION ENGINEER:
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(925) 314-7910

CODE CONSULTANT:
ARS & ASSOCIATES
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(415) 333-8091

REV.	DESCRIPTION	DATE
	FOUNDATION PERMIT	05/26/2006
1	ADDENDUM 2 STRUCTURE	11/16/2006
	ADDENDUM 2 REVISIONS	03/06/2006
2	ADD2 - REV2 PEER REVIEW	05/26/2006
3	ADD2-REV3 BAUGRID	09/25/2006

DESIMONE
160 SANSOME STREET 16TH FLOOR
SAN FRANCISCO, CA 94104 3722
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STAMP:
REGISTERED PROFESSIONAL ENGINEER
NO. S 4827
EXP. 09-30-07
STRUCTURAL
STATE OF CALIFORNIA

SCALE: AS NOTED
DRAWN BY:
PROJECT #:

PROJECT TRUE

VERTICAL SYSTEMS
TOWER MOMENT FRAME
SCHEDULE AND DETAILS

S3-2.71

DRAWING INDEX

DWG. NO.	DRAWING TITLE	FOUNDATION PERMIT 05/24/2005	ADDENDUM 2 - STRUCTURE 11/18/2005	CITY COMMENTS 01/3/2006	ADDENDUM 2-REVISIONS 03/08/2006	ADDENDUM 2 - REVISION 2 PEER REVIEW 05/26/2006	ADDENDUM 2 - REVISION 3 BAUGRID 09/25/2006
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S0-0 16	STRUCTURAL REVIEW LETTERS	●	●	●	●	●	
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S2-0 B4 11	LEVEL B4 PLAN	●	●		●		
S2-0 B3 11	LEVEL B3 PLAN	●	●		●		
S2-0 B2 11	LEVEL B2 PLAN	●	●		●		
S2-0 B1 01	LEVEL B1 - REFERENCE PLAN	●	●		●		
S2-0 B1 11	LEVEL B1 WEST PLAN	●	●		●	●	
S2-0 B1 12	LEVEL B1 WEST PLAN BOTTOM REINFORCEMENT	●	●		●		
S2-0 B1 13	LEVEL B1 WEST PLAN TOP REINFORCEMENT	●	●		●		
S2-0 B1 14	LEVEL B1 WEST PLAN PILE LOCATION		●		●		
S2-0 B1 21	LEVEL B1 EAST PLAN	●	●		●		
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S2-0 01 11	LEVEL 1 WEST PLAN	●	●		●	●	
S2-0 01 21	LEVEL 1 EAST PLAN	●	●		●		
S2-0 02 01	LEVEL 2 - REFERENCE PLAN	●	●		●		
S2-0 02 11	LEVEL 2 WEST PLAN	●	●		●	●	
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S2-0 02 21	LEVEL 2 EAST PLAN	●	●		●		
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S2-0 03 11	LEVEL 3 WEST PLAN	●	●		●	●	
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S2-1 09 01	LEVELS 9, 18 PLAN	●	●		●	●	
S2-1 10 01	LEVEL 10, 11, 19, 20 PLAN				●		
S2-1 13 01	LEVELS 13, 22 PLAN	●	●		●		
S2-1 25 01	LEVEL 25 PLAN	●	●		●		
S2-1 26 01	LEVEL 26 PLAN	●	●		●		
S2-1 27 01	LEVEL 27 PLAN	●	●		●		
S2-1 28 01	LEVELS 28-41 PLAN		●		●		
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S2-1 43 01	LEVEL 43 PLAN	●	●		●		
S2-1 44 01	LEVEL 44 PLAN				●		
S2-1 46 01	LEVEL 46 PLAN	●	●		●		
S2-1 47 01	LEVEL 47 PLAN	●	●		●		
S2-1 48 01	LEVELS 48-56 PLAN	●	●		●		
~~[illegible]~~	~~[illegible]~~	●					
S2-1 57 01	LEVEL 57 PLAN	●	●		●		
S2-1 58 01	LEVEL 58 PLAN	●	●		●		
S2-1 59 01	LEVEL 59 PLAN	●	●		●		
S2-1 60 01	LEVEL 60 PLAN	●	●		●		
S2-1 61 01	LEVEL 61 PLAN	●	●		●		
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S2-2 10 01	LEVEL 10 PLAN				●		
S2-2 11 01	LEVEL 11 PLAN				●		
S2-2 12 01	LEVEL 12 PLAN	●	●		●		
S2-2 13 01	LEVEL 13 PLAN	●			●		
S2-2 14 01	LEVEL 14 PLAN		●		●		
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S3-1 11	FOUNDATION SECTIONS AND DETAILS	●	●		●		
S3-1 12	FOUNDATION SECTIONS AND DETAILS	●	●		●		
S3-1 13	FOUNDATION SECTIONS AND DETAILS	●	●		●		
S3-1 14	FOUNDATION SECTIONS AND DETAILS	●	●		●		
S3-1 15	FOUNDATION SECTIONS AND DETAILS	●	●		●		
S3-2	VERTICAL SYSTEMS						
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S3-2 02	TOWER COLUMN DETAILS	●	●		●	●	●
S3-2 03	TOWER COLUMN DETAILS				●		
S3-2 08	PODIUM/OFFICE/AMENITIES COLUMN SCHEDULE	●	●		●		

DWG. NO.	DRAWING TITLE	FOUNDATION PERMIT 05/24/2005	ADDENDUM 2 - STRUCTURE 11/18/2005	CITY COMMENTS 01/3/2006	ADDENDUM 2-REVISIONS 03/08/2006	ADDENDUM 2 - REVISION 2 PEER REVIEW 05/26/2006	ADDENDUM 2 - REVISION 3 BAUGRID 09/25/2006
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S3-2 13	PODIUM SHEAR WALL ELEVATIONS	●	●		●		
S3-2 21	TOWER SHEAR WALL SCHEDULE	●	●		●	●	
S3-2 23	TOWER LINK BEAM SCHEDULE AND DETAILS	●	●		●	●	
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S3-2 32	TOWER SHEAR WALL PLANS	●	●		●		
S3-2 33	TOWER SHEAR WALL PLANS	●	●		●		
S3-2 34	TOWER SHEAR WALL PLANS	●	●		●		
S3-2 35	TOWER SHEAR WALL PLANS	●	●		●		
S3-2 36	TOWER SHEAR WALL PLANS	●	●		●		
S3-2 37	TOWER SHEAR WALL PLANS		●		●		
S3-2 38	TOWER SHEAR WALL PLANS		●		●		
S3-2 39	TOWER SHEAR WALL PLANS		●		●		
S3-2 41	PODIUM SHEAR WALL PLANS	●	●		●	●	
S3-2 42	PODIUM SHEAR WALL PLANS	●	●		●	●	
S3-2 43	PODIUM SHEAR WALL PLANS	●	●		●		
S3-2 51	TOWER SHEAR WALL DETAILS	●	●		●		●
S3-2 52	TOWER SHEAR WALL DETAILS	●	●		●		●
S3-2 53	TOWER SHEAR WALL DETAILS	●	●		●		●
S3-2 54	TOWER SHEAR WALL DETAILS	●	●		●		●
S3-2 55	TOWER SHEAR WALL DETAILS	●	●		●		●
S3-2 56	TOWER SHEAR WALL DETAILS		●		●		●
S3-2 57	TOWER SHEAR WALL DETAILS				●	●	
S3-2 58	TOWER SHEAR WALL DETAILS	●	●		●	●	
S3-2 59	TOWER SHEAR WALL DETAILS		●		●	●	
S3-2 71	TOWER MOMENT FRAME SCHEDULE AND DETAILS	●	●		●	●	●
S3-2 72	TOWER MOMENT FRAME DETAILS	●	●		●	●	
S3-2 73	TOWER MOMENT FRAME DETAILS	●	●		●	●	
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S3-3 02	TYPICAL CONCRETE DETAILS	●	●		●		
S3-3 03	TYPICAL CONCRETE DETAILS	●	●		●		
S3-3 11	SUPERSTRUCTURE SECTIONS AND DETAILS	●	●		●		
S3-3 12	SUPERSTRUCTURE SECTIONS AND DETAILS	●	●		●		
S3-3 13	SUPERSTRUCTURE SECTIONS AND DETAILS	●	●		●		
S3-3 14	SUPERSTRUCTURE SECTIONS AND DETAILS	●	●		●		
S3-3 15	SUPERSTRUCTURE SECTIONS AND DETAILS	●	●		●		
S3-3 16	SUPERSTRUCTURE SECTIONS AND DETAILS		●		●		
S3-3 17	SUPERSTRUCTURE SECTIONS AND DETAILS		●		●		
S3-3 18	SUPERSTRUCTURE SECTIONS AND DETAILS		●		●		
S3-3 21	TYPICAL POST-TENSION SECTIONS AND DETAILS	●	●		●		
S3-3 22	TYPICAL POST-TENSION SECTIONS AND DETAILS	●	●		●	●	

301 MISSION STREET

OWNER:
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FIRE PROTECTION ENGINEER:
HUGHES ASSOCIATES
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SAN RAMON, CA 94583

CODE CONSULTANT:

REV.	DESCRIPTION	DATE
	FOUNDATION PERMIT	05/24/2005
	ADDENDUM-2 STRUCTURE	11/18/2005
	ADDENDUM 2 REVISIONS	03/08/2006
	ADD2-REV2 PEER REVIEW	05/26/2006
	ADD2-REV3 BAUGRID	09/25/2006

DESIMONE
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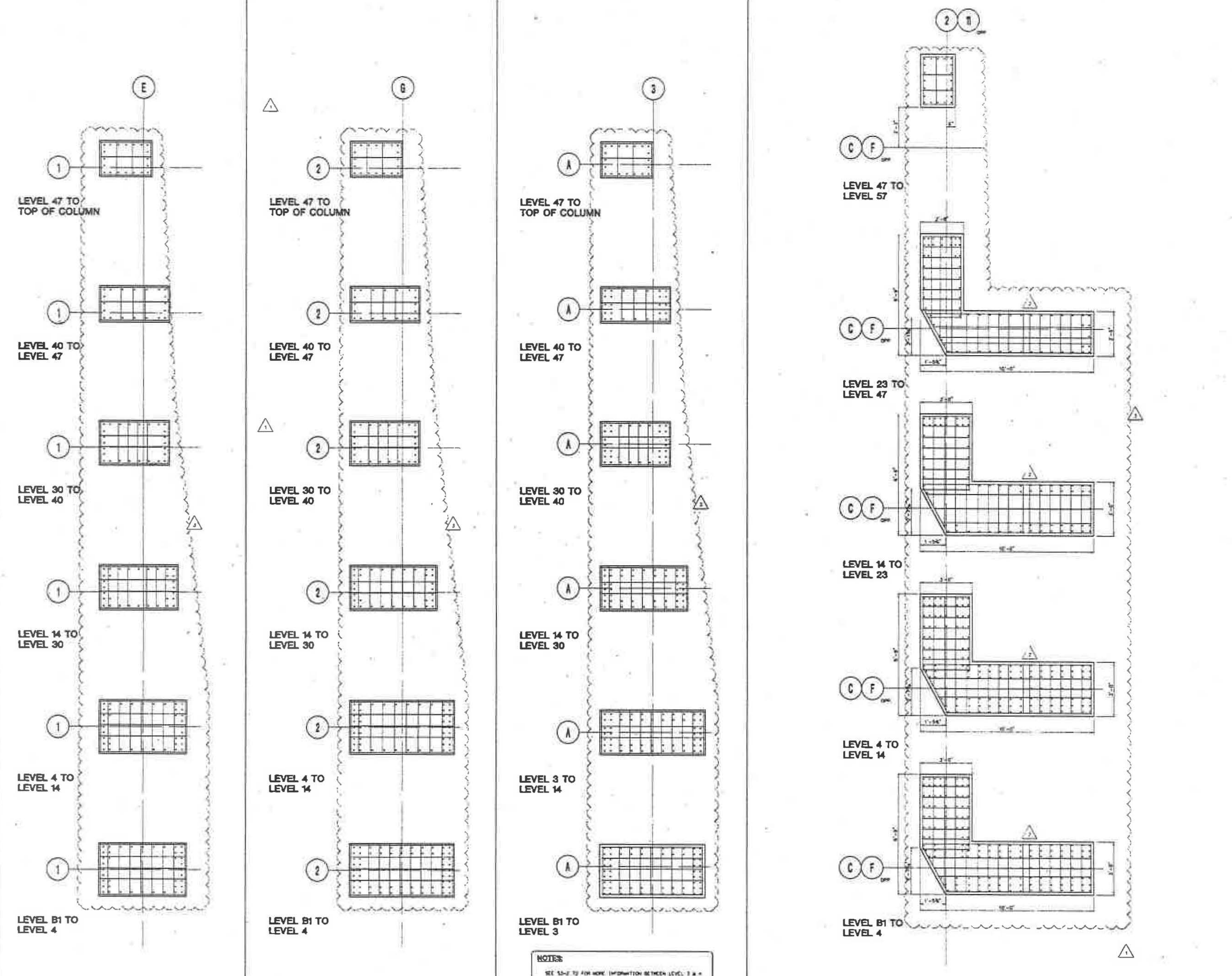
STAMP:

REGISTERED PROFESSIONAL ENGINEER
STRUCTURAL
STATE OF CALIFORNIA

SCALE: N.T.S.
DRAWN BY:
PROJECT #:

GENERAL INFORMATION
DRAWING INDEX

S0-0.10



NOTES:
SEE S3-2 TO FOR MORE INFORMATION BETWEEN LEVEL 3 & 4

15 COLUMN A DETAILS
SCALE: 1/2"=1'-0"

14 COLUMN C DETAILS
SCALE: 1/2"=1'-0"

13 COLUMN D DETAILS
SCALE: 1/2"=1'-0"

12 COLUMN B DETAILS
SCALE: 1/2"=1'-0"

301 MISSION STREET

OWNER:
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REV.	DESCRIPTION	DATE
	FOUNDATION PERMIT	05/24/2005
1	ADDENDUM 2 STRUCTURE	11/18/2005
	ADDENDUM 2 REVISIONS	03/06/2006
2	ADD2 - REV2 PEER REVIEW	05/26/2006
3	ADD2-REV3 BAUGRID	09/25/2006

DESIMONE
160 SANSOME STREET 16TH FLOOR
SAN FRANCISCO CA 94104 3722
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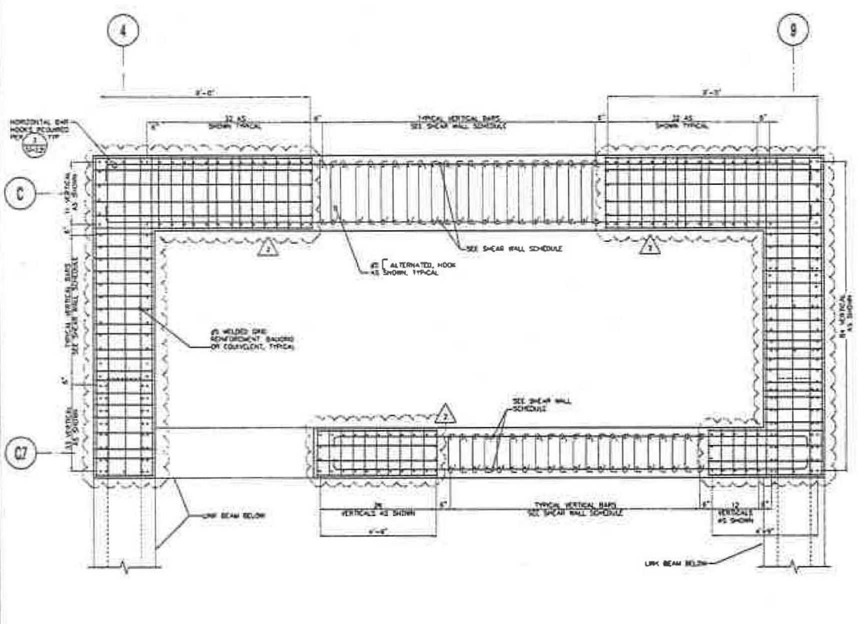
STAMP

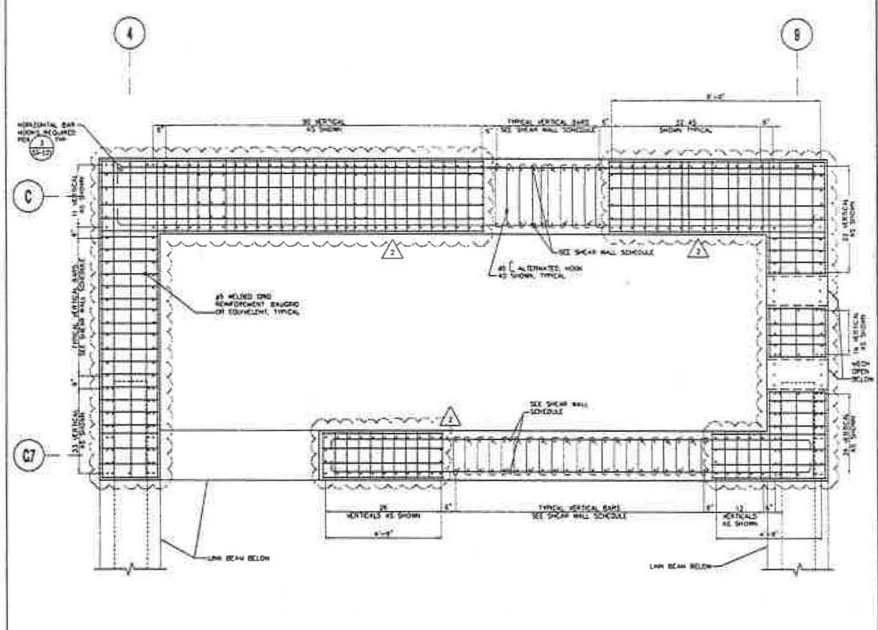
SCALE: AS NOTED
DRAWN BY: H/H
PROJECT #: 4065

VERTICAL SYSTEMS
TOWER COLUMN DETAILS

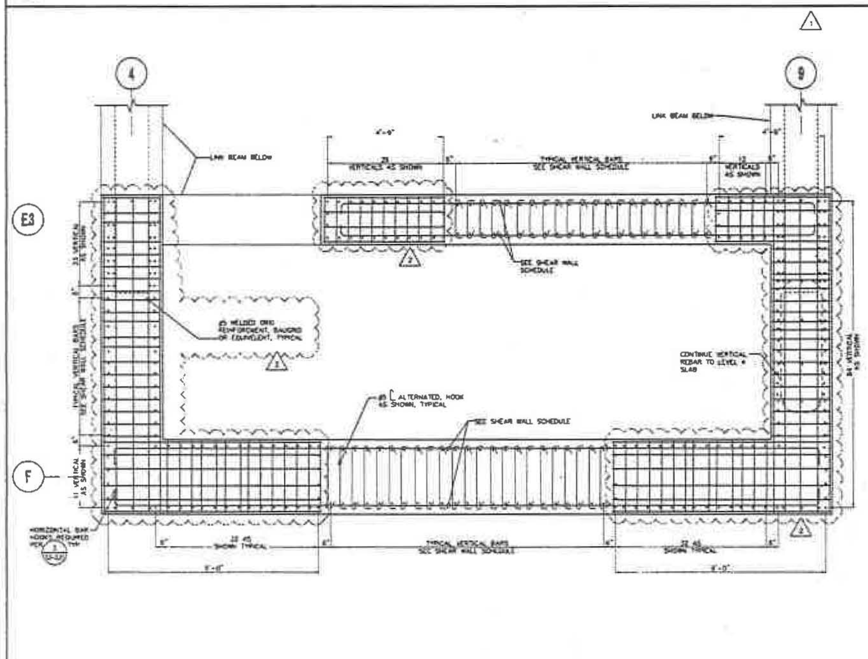
S3-2.02



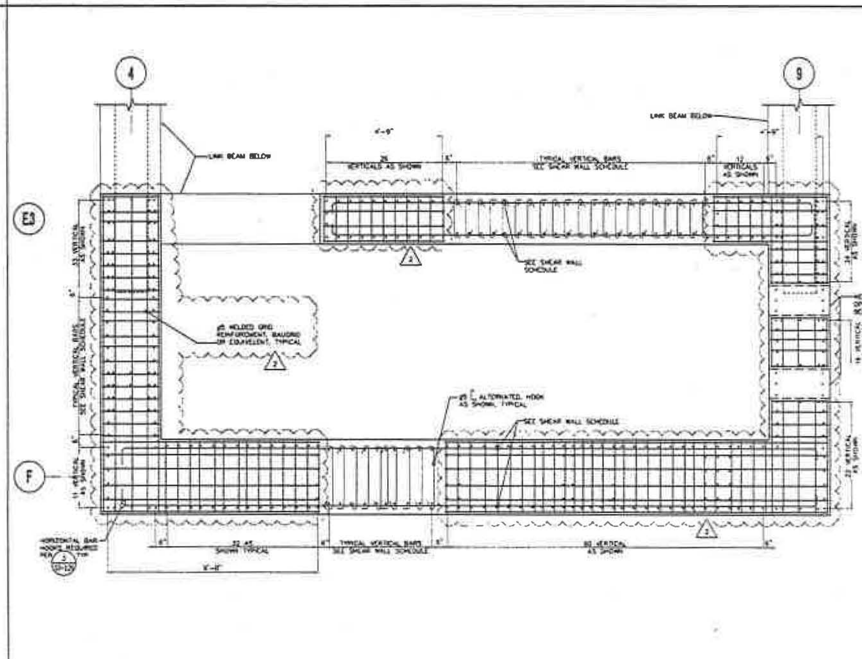
2 SHEAR WALL DETAIL
SCALE: 1/2"=1'-0"



1 SHEAR WALL DETAIL - LEVEL 02
SCALE: 1/2"=1'-0"



4 SHEAR WALL DETAIL
SCALE: 1/2"=1'-0"



3 SHEAR WALL DETAIL - LEVEL 02
SCALE: 1/2"=1'-0"

301 MISSION STREET

OWNER:
MISSION STREET DEVELOPMENT PARTNERS LLC 735 MARKET STREET
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REV.	DESCRIPTION	DATE
	FOUNDATION PERMIT	05/24/2005
1	ADDENDUM 2 STRUCTURE	[illegible]
	ADDENDUM 2 REVISIONS	[illegible]
2	ADD2-REV3 BAUGRID	[illegible]

DESIMONE
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STAMP

SCALE: AS NOTED
DRAWN BY: ELS
PROJECT #: 4069

VERTICAL SYSTEMS
SHEAR WALL DETAILS

S3-2.52

Concrete. Concrete strengths in the Tower walls and frames vary between 7 and 10 ksi, and in the Mid-rise between 7 and 8 ksi. All floor slabs are 5 ksi.

Reinforcing. The shear walls in both buildings and the moment frames in the Tower use Grade 75 reinforcing for bars larger than #8's. All shear wall confinement steel is Grade 75 for areas where the concrete strength is 8 ksi and higher. Steel of all grades used as part of the lateral system must meet the ductility requirements of ASTM A706.

Foundations

Tower. The Tower foundation consists of a 10-foot thick pile cap supported by pre-cast concrete piles. The bottom of the pile cap is approximately 25' below the existing grade. The initial vertical pile displacement due to slippage required to fully engage the pile is expected to be approximately 1" by the time of project construction completion. Additional long-term pile settlement due to compression of the underlying clay layers is expected to be as much as 5". As the piles are only located directly below the Tower footprint, this settlement is expected to occur uniformly over the Tower foundation area.

Mid-rise. The Mid-rise structure rests on a mat foundation that varies between 6 feet and 8 feet in thickness. The bottom of this excavation is approximately 63 feet below the existing grade. Tie-downs resist hydrostatic uplift pressures under the portion of the deep excavation that is not directly below the Mid-rise, i.e., the area between the Mid-rise and the Tower.

Building Separation

The foundations and lateral systems of the two buildings are considered completely separate because a joint is located between them at the B1, Ground, 2nd, and 3rd Floors. "Hinge slabs" allow circulation between the two buildings, while still accommodating differential settlement and seismic displacements between the two structures.

Wind Loads

A wind tunnel study was performed and a report issued by Rowan Williams Davies & Irwin Inc. (RWDI). The results of the report were used to evaluate both the Tower and Mid-rise. Wind does not control either design forces or interstory drifts for either structure.

Seismic Loads

Site-specific ground motions provided by the geotechnical engineer of record, Treadwell and Rollo, were used for the analyses of both structures. Earthquake design forces acting on individual elements were obtained by performing response spectrum analyses with the proprietary computer program "ETABS" written by Computers and Structures, Inc. of Berkeley, California.

center of mass.

Mid-rise. Due to the eccentricity of the shear walls relative to the center of mass of the building, I Mid-rise exhibits a slight torsional irregularity. For this reason the base shear was not reduced accordance with 1631.5.4.2.

Different base shears were used for checking design forces and building interstory drifts. (Since period of the structure is relatively short, the minimum base shear equations of 30-6 and 30-7 do apply.)

Design Procedures

All elements of the structure are designed and detailed in accordance with the load combinations requirements of the 2001 SFBC. Additional procedures were also followed as listed below.

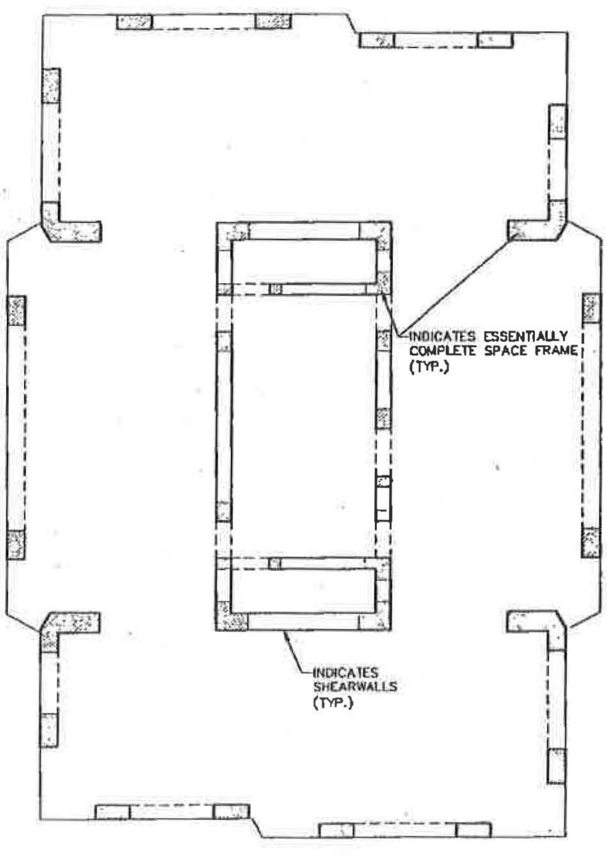
Steel Link Beams. The 2001 SFBC does not address the steel link beams used within the core of Tower. These elements are designed using the 2002 AISC Seismic Provisions requirements for Spe Reinforced Concrete Shear Walls Composite with Structural Steel Elements.

Capacity Design. Each of the 12 outriggers connecting to the central shear core of the Tower cont two diagonally reinforced link beam elements. These links are designed to remain elastic under code-prescribed seismic loads, but it is desirable for them to yield first once the design loads exceeded by a major earthquake. In order to insure this behavior, the capacities of the link be were calculated and increased by an overstrength factor. The resulting forces were used as demands for which the following elements were designed: the portion of each outrigger connectin the core walls, the outrigger columns, and the pile cap.

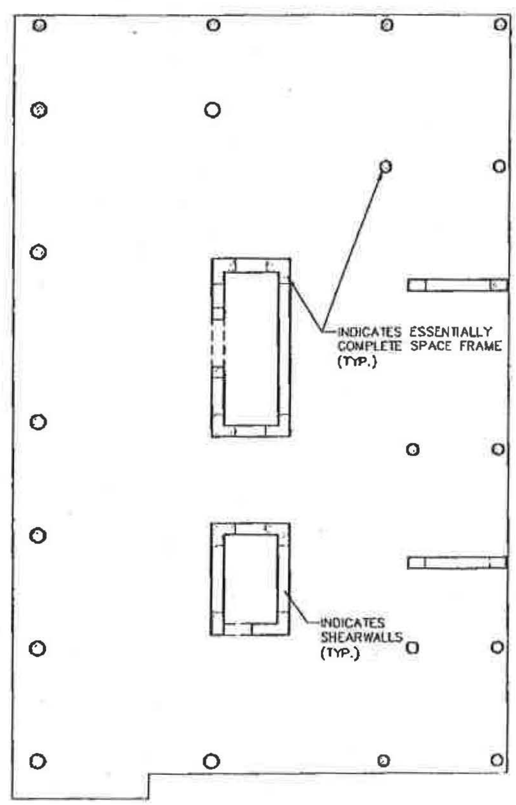
Note that this approach is not required by the SFBC and represents an effort to "go beyond the co This increases our confidence that in a large earthquake the very ductile link beam elements wil first, and the critical connecting elements of the structure will remain essentially undamaged. design of all elements still meets the requirements of the SFBC.

The outriggers columns are designed to remain elastic when simultaneously subjected to the cap of all link beams, as well as all tributary gravity loads.

The pile cap under the Tower is designed to remain elastic when subjected to the capacities o outrigger columns, as well as the expected maximum moment at the base of the shear wall core.



TOWER



MID-RISE

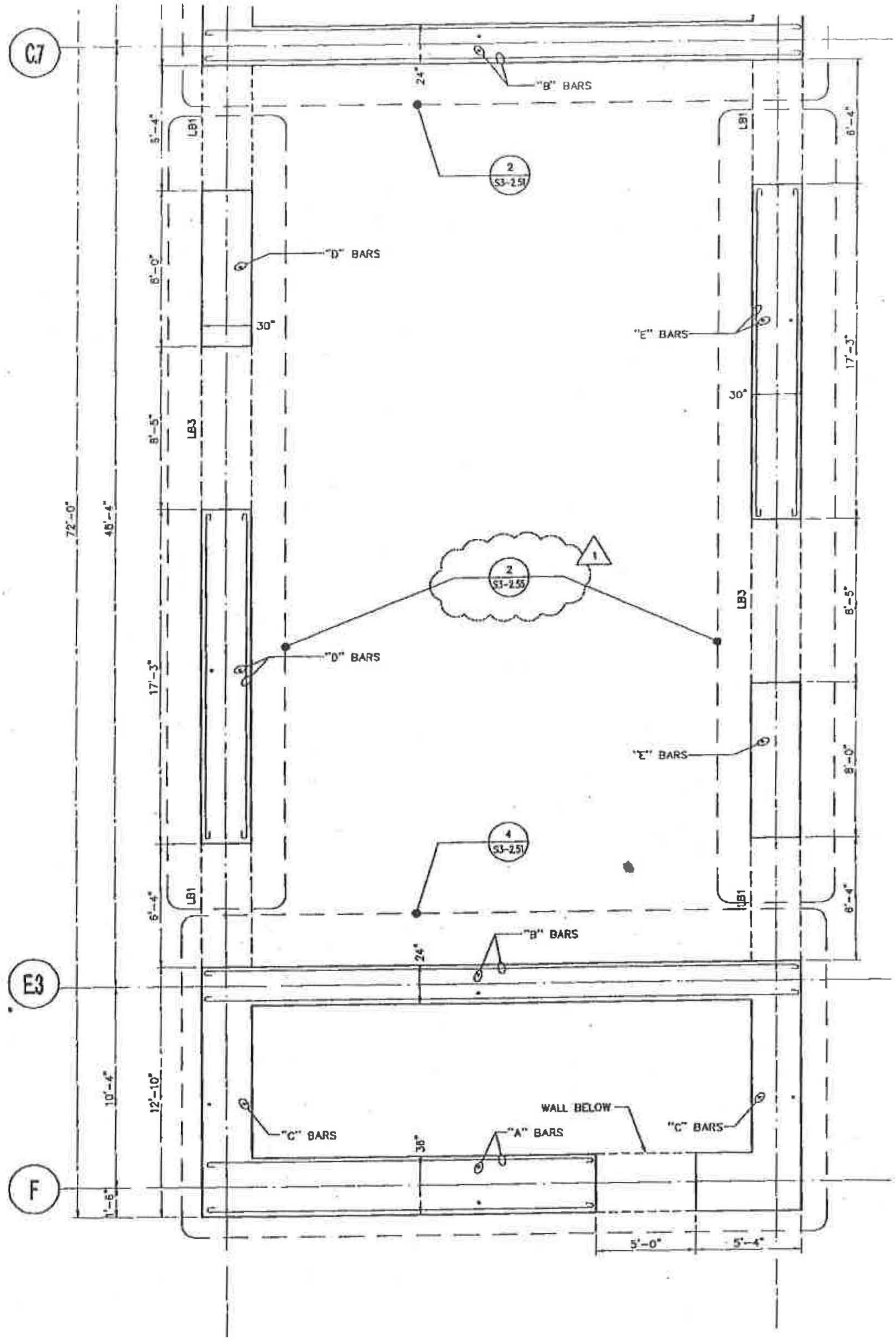
NOTES:

TOWER:

AN ESSENTIALLY COMPLETE SPACE FRAME PER UBC 1629.6.5 IS PROVIDED BY SPECIALLY REINFORCED MOMENT FRAME COLUMNS AND THE BOUNDARY ELEMENTS OF THE SHEARWALLS. THE CORE SHEARWALLS MAKE UP A CONTINUOUS GRAVITY LOAD RESISTING SPACE FRAME.

MID-RISE:

GRAVITY LOAD IS RESISTED BY THE COLUMNS SURROUNDING THE CORE AND THE BOUNDARY ELEMENT COLUMNS EMBEDDED IN THE SHEARWALLS. THESE ELEMENTS
... SPACE FRAME NEEDED TO CLASSIFY THE



NOTES:

1. SEE SHEAR WALL SCHEDULE FOR SIZE AND SPACING OF TYPICAL BARS.
2. TYPICAL BARS APPLY TO E.F. OF WALL.

2 SHEAR WALL PLAN - LEVEL 01

SCALE: 1/4" = 1'-0"

WALL REINFORCEMENT SCHEDULE

BUILDING LEVEL		A BARS	B BARS	C BARS	D BARS	E BARS
56–61	VERTICAL	#7@12"	#7@12"	#7@12"	#7@12"	#7@12"
	HORIZONTAL	#7@8"	#7@12"	–	#7@12"	#7@12"
52–56	VERTICAL	#7@12"	#7@12"	#7@12"	#7@12"	#7@12"
	HORIZONTAL	#7@8"	#7@12"	–	#7@12"	#7@12"
48–52	VERTICAL	#7@12"	#7@12"	#7@12"	#7@12"	#7@12"
	HORIZONTAL	#7@8"	#7@12"	–	#7@12"	#7@12"
44–48	VERTICAL	#7@9"	#7@9"	#7@9"	#7@12"	#7@12"
	HORIZONTAL	#7@8"	#7@12"	–	#7@12"	#7@12"
40–44	VERTICAL	#7@9"	#7@9"	#7@9"	#7@12"	#7@12"
	HORIZONTAL	#7@4"	#7@12"	–	#7@12"	#7@6"
36–40	VERTICAL	#7@9"	#7@9"	#7@9"	#7@12"	#7@12"
	HORIZONTAL	#7@4"	#7@12"	–	#7@8"	#7@6"
32–36	VERTICAL	#7@9"	#7@9"	#7@9"	#7@12"	#7@12"
	HORIZONTAL	#7@4"	#7@12"	–	#7@8"	#7@6"
28–32	VERTICAL	#7@9"	#7@9"	#7@9"	#7@12"	#7@12"
	HORIZONTAL	#7@4"	#7@12"	–	#7@8"	SEE NOTE 6
24–28	VERTICAL	#8@6"	#8@6"	#8@6"	#7@12"	#7@12"
	HORIZONTAL	#7@4"	#7@12"	–	#7@6"	#8@8"
20–24	VERTICAL	#8@6"	#8@6"	#8@6"	#7@12"	#7@12"
	HORIZONTAL	#7@4"	#7@12"	–	#7@6"	#8@8"
16–20	VERTICAL	#8@6"	#8@6"	#8@6"	#7@6"	#7@6"
	HORIZONTAL	#8@4"	#7@12"	–	#7@6"	#8@8"
12–16	VERTICAL	#8@6"	#8@6"	#8@6"	#7@6"	#7@6"
	HORIZONTAL	#8@4"	#7@12"	–	#7@4"	#8@8"
08–12	VERTICAL	#8@6"	#8@6"	#8@6"	#7@6"	#7@6"
	HORIZONTAL	#8@4"	#7@12"	–	#8@4"	#8@8"
04–08	VERTICAL	#8@6"	#8@6"	#8@6"	#7@6"	#7@6"
	HORIZONTAL	#8@4"	#7@12"	–	#8@4"	#8@8"
01–04	VERTICAL	#9@6"	#9@6"	#9@6"	#8@6"	#8@6"
	HORIZONTAL	#8@4"	#8@8"	–	#8@4"	#8@4"
B1–01	VERTICAL	#9@6"	#9@6"	#9@6"	#8@6"	#8@6"
	HORIZONTAL	#8@4"	#8@8"	–	#8@4"	#8@4"

1

NOTES:

1. CONCRETE STRENGTHS ARE:
   10 ksi B1 – L20
   8 ksi L21 – L40
   7 ksi L41 – ROOF
2. fy = 75 ksi (#9 AND HIGHER), fy = 60 ksi (#8 AND LOWER)
3. USE fy = 75 ksi FOR ALL TIES.
4. ALL BARS E.F., U.O.N.
5. ALL SPACING IS CENTER-TO-CENTER. SPACING IS TYPICAL U.O.N.
6. #8@6" DOWN TO LEVEL 27 SLAB

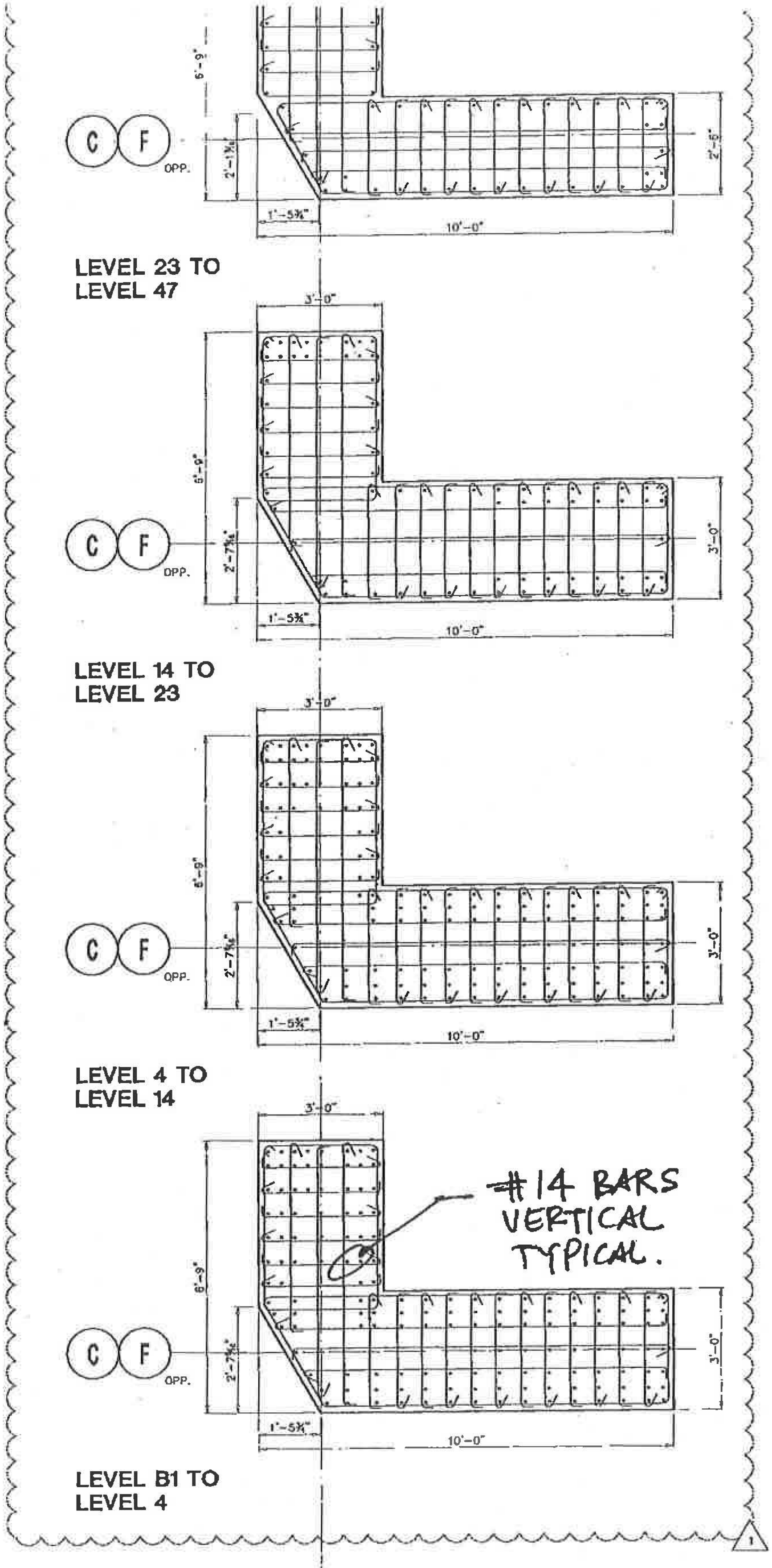
1

7 SHEAR WALL SCHEDULE

SCALE: N.T.S.

REV.

1



12 COLUMN B DETAILS

SCALE: 1/2"=1'-0"

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(925) 314-7910

CODE CONSUL
ARS & ASSOCIATES
301 JUNIPERO SERF
SAN FRANCISCO, CA
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REV.

1

DI
160 SAN
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STAMP:

SCALE:
DRAWN BY:
PROJECT #:

VERTICA

TOWEI

S3-2

301 MISSION STREET, SAN FRANCISCO

PEER REVIEW

1. The L-shaped columns will be in torsion for frame action along axis 2 and axis 11. Consider torsion for design.

   We disagree with your simple response. Please provide detailed calculations that account for eccentricity between the center of resistance of column and outriggers and frame beams.

2. The L-shaped columns support outriggers of the prime lateral system. It should be shown that participation or failure of the more rigid element will not impair the vertical and lateral resisting ability of the gravity load and lateral moment resisting system. (See section 1633.2.4.1).

   Our intent here is that the backup moment frame should not be impaired by the failure of outriggers or shearwalls. Please provide detailed calculations to demonstrate that Moment Frame will be able to take its demand once the shearwalls have failed.

3. Low-rise mat show 69 psf reinforce for total area. It looks excessive. (It is #11 @ 4.3" E.W. T & B for 8' mat.)

   Resolved.

4. At one side of shear wall at line D.5, a ramp that has an opening in the diaphragm. Clarify how the shear will travel to both basement walls at A.1 and K.

   Resolved.

5. Verify by calculations that ground floor diaphragm behave as a rigid diaphragm transferring forces to the perimeter basement walls and to the core. Possible reverse shear might happen in the basement and in the core walls below.

   We disagree with your response. We believe that there will be reverse shear and floor needs to be modeled to account for it or properly detailed that it is not connected with the shear walls. Please provide detailed calculations as requested above or floor to wall connection details.

6. The mid-rise and the high-rise towers are joined at the ground floor and B1 levels. The high-rise tower has mat with piles more rigid than mid-rise 5-story basement. Verify deformation compatibility and amount of base shear that will be resisted by piles.

   We are generally in agreement with your approach but we would like to get the calculations for lateral loads on piles and any horizontal movement that occurs from the lateral load.

7. There are shear walls surrounded by openings at both sides. Verify collectors requirements to deliver shear to these walls.

   We agree with your response but would like to have calculations for at least ground floor level.

8. Settlement compatibility between high-rise on piles and mid-rise on mat footing total settlement for both could be different, but there is ground floor slab without a joint that could get cracked.

   Resolved.

9. At 9' deep mat on piles, how is the modulus of subgrade reaction applied to pile footings.

   Resolved.

10. Is 9' deep pile cap required in full building area? There are areas where depth could be greatly reduced. (K-H for example)

    Resolved.

11. The differential shortening in columns and walls will produce additional significant moments on outrigger beams. Is there a mechanism to relieve them from these forces?

    We disagree with your response. Please provide detailed stress calculations (moments, shears) that account for shortening of all vertical members.

12. Optimize P/T slab thickness at all locations.

    This item should be reviewed with the contractor for cost impact.

13. Main tower moment frames are all single bay frames that are not effective. Some of the bays can't be considered as a frame because clear span to depth is less than 4 – for example B0403.

    We disagree, for example check the span to depth ratio of beam B3 on third floor.

14. Please provide design criteria for outrigger beams. Are they designed as a "deep beam" with a consideration for non-linear strain distribution. What forces will be considered for designing columns that get forces from outriggers?

    The capacity of the frame columns should be more than the capacity of outrigger or omega x outrigger forces. Please provide the capacity of the outrigger using non-linear failure analysis of outrigger + shear walls.

15. a. Column transfer at 2nd floor line H with sloped column at 1st floor will create additional lateral component on both levels that will require beams and slab between frames to be designed for additional axial force.

    Please provide calculations when this design is finalized.

    b. Very deep column section – 26' deep will act as a shear wall and attract a lot of additional seismic load to this frame. Careful considerations should be taken to design this transfer column for all applied loads.

    This member does not qualify as frame member. It should be properly modeled in ETABS and designed for omega x seismic forces. Additionally, Beam at level 1 should comply with UBC 1921.3.1.1.

c. Sloped column should be included in the building model.

We agree with your response in concept. Please submit the properties of the sloped column that were used in the ETABS model.

16. There are 4 or 5 different round column sizes on one level – ground level mid-rise. Please verify if unification of sizes is possible to reduce cost.

Resolved.

17. a. Design criteria on drawings describes dual system, shear wall with SMRF, and R = 8.5. Mid-rise building has no SMRF. This building also has vertical structural irregularities such as discontinuous shear wall that should be considered.

Resolved.

b. Code equations 30-6 and 30-7 need not be considered for drift check.

Resolved.

c. Drift check should include accidental torsion.

Resolved.

18. Please specify wind load design criteria for strength and for drift. Compare wind load and seismic.

Resolved.

19. All outriggers are unusual in shape and can't be clearly designed as a deep beams or discontinuous shear walls. Based on their importance for overall stability of the building non-linear time history analysis should be performed to investigate performance of these important elements and bring factor of safety for them to a desirable level.

We reserve our response to this comment till we see the response to comment 14 above.

20. Provide design calculations and details that account for P/T slab shortening due to concrete shrinkage.

21. Please provide the detailed design and analysis of W14 steel link beams.

22. Please submit the ETABS model and backup calculations justifying cracked section properties.

23. Please provide calculations for diaphragm design.

NOTES:

1. TESTS SHALL BE PERFORMED FOR BOTH #4 AND #5 BAUGRIDS AS SHOWN ON PAGE 29 OF THE BAUGRID QUALITY CONTROL MANUAL (REPRODUCED HERE FOR CLARITY.) ALL APPLICABLE ASTM PROCEDURES AND/OR THE BAUGRID QUALITY CONTROL MANUAL SHALL BE ADHERED TO.
2. COLUMN TEST BAUGRID FABRICATION SHOULD ONLY PROCEED IF BAUGRID COUPONS TESTS ARE SATISFACTORY.
3. TESTS SHALL BE PERFORMED IN THE MANUFACTURING FACILITY IN CHINA USING THE SAME TESTING APPARTUS AS USED FOR THE 301 MISSION PROJECT.
4. ALL TESTS SHALL BE PERFORMED WITH SMITH–EMERY AS WITNESS. SMITH–EMERY SHALL PRODUCE A REPORT PRESENTING ALL TESTING RESULTS AND A STATEMENT AS TO THE TESTS CONFORMANCE WITH THE BAUGRID QUALITY CONTROL MANUAL. ALL RAW STRESS–STRAIN DATA SHALL ALSO BE INCLUDED IN THE REPORT.

PROJECT: 301 MISSION	JOB #: 4069	SCALE: N.T.S.
TITLE: BAUGRID COUPON TEST SETUP	DATE: 11/03/2006	DWG. NO.
DESIMONE 160 SANSOME STREET 16TH FLOOR SAN FRANCISCO, CA 94104 - 3722 T. 415.398.5740 F. 415.398.9834	DRAWN: NJR CHECKED: DDR, RMP	SK-02

PROCEDURE:

1. TEST SPECIMENS
   a. SHAKEDOWN TEST. SPECIMENS A1, A2, & A3. THREE (3) SPECIMENS CONTAINING BAUGRID WILL BE BUILT PER SK-01.
   b. CITY TEST. SPECIMENS B1, B2, & B3. THREE (3) SPECIMENS CONTAINING BAUGRID WILL BE BUILT PER SK-01.

2. TEST INSTRUMENTATION
   a. ALL SPECIMENS WILL BE INSTRUMENTED WITH TWO (2) AXIAL STRAIN MEASUREMENT DEVICES (LVDT'S) ON THE EXTERIOR AND ON OPPOSITE SIDES OF THE SPECIMEN ACROSS THE TESTING REGION.
   b. ALL SPECIMENS WILL BE INSTRUMENTED WITH TWO (2) STRAIN GAGES ON THE LONGITUDINAL REINFORCEMENT. THESE GAGES WILL BE PLACED ON OPPOSITE SIDES OF THE SPECIMEN, NEAR THE LVDT'S, WITHIN THE TESTING REGION.
   c. ALL SPECIMENS WILL BE INSTRUMENTED WITH FOUR (4) STRAIN GAGES ON THE TRANSVERSE REINFORCEMENT. STRAIN GAGES ON THE TRANSVERSE BAUGRID REINFORCEMENT WILL BE PLACED AS CLOSE AS POSSIBLE TO THE WELDS.

3. PURPOSE OF EACH TEST
   a. SPECIMENS A1, A2, & A3 WILL BE TESTED WHEN THE CONCRETE STRENGTH HAS REACHED 8,000 PSI. THE PURPOSE OF THESE TESTS WILL BE TO MAKE SURE THE TESTING PROCEDURE IS UNDERSTOOD PRIOR TO TESTING THE CITY TEST SPECIMENS.
   b. SPECIMENS B1, B2, & B3 WILL BE TESTED WHEN THE CONCRETE STRENGTH HAS REACHED 10,000 PSI. THE OUTCOME OF THESE TESTS WILL DETERMINE IF BAUGRID IS ACCEPTABLE FOR USE ON THE 301 MISSION STREET PROJECT.

4. CONCRETE STRENGTH TESTS
   a. FORTY (40) CONCRETE CYLINDERS SHALL BE TAKEN FROM THE SAME CONCRETE USED FOR THE TEST SPECIMENS.
   b. TWO (2) CYLINDERS SHALL BE BROKEN ON THE 5TH DAY AFTER CONCRETE PLACMENT AND ON EACH DAY THEREAFTER UNTIL THE CONCRETE REACHES 10,000 PSI, WHICH IS EXPECTED AT APPROXIMATELY FOURTEEN (14) DAYS AFTER PLACEMENT.
   c. TWO (2) ADDITIONAL CYLINDERS SHALL BE BROKEN AT 28, 56, AND 90 DAYS AFTER CONCRETE PLACEMENT.

5. TESTING PROCEDURE
   a. EACH SPECIMEN WILL BE SUBJECTED TO MONOTONIC CONCENTRIC LOADING. (THE APPROPRIATE RATE OF LOADING IS TO BE DETERMINED AND AGREED TO PRIOR TO TESTING.)
   b. SPECIMENS A1, A2, AND A3 WILL BE LOADED UNTIL FAILURE.
   c. SPECIMENS B1, B2, AND B3 WILL BE LOADED UNTIL THEY HAVE REACHED THE ACCEPTANCE CRITERIA ONLY. ADDITIONAL LOADING MAY BE APPLIED AT THE OWNER'S SOLE DISCRETION.

6 .TEST ACCEPTABILITY CRITERIA
   a. CITY TEST SPECIMENS B1, B2, & B3. EACH TEST WILL BE DEEMED SUCCESSFUL IF THE AVERAGE OF THE TWO AXIAL STRAIN DEVICES REACHES 0.6%.

7. BAUGRID EQUIVALENCY TESTS
   a. BAUGRID QUALITY CONTROL TESTS 1-5 AS SHOWN IN SK-02 WILL BE PERFORMED ON THE #4 BAUGRIDS USED IN THE TEST SPECIMENS, AS WELL AS REPRESENTATIVE #5 BAUGRIDS TO THOSE BEING USED AT THE 301 MISSION STREET PROJECT.
   b. SUCCESSFUL COMPLETION OF THESE TESTS WILL BE DEMONSTRATED IF THE #4 BAUGRIDS AND #5 BAUGRIDS ALL PASS THE ASSOCIATED ASTM AND BAUGRID QUALITY CONTROL MANUAL CRITERIA.

PROJECT: 301 MISSION	JOB #: 4069	SCALE: N.T.S.
TITLE: BAUGRID COLUMN TEST SETUP PROCUDURES	DATE: 11/03/2006	DWG. NO.
DESIMONE 160 SANSOME STREET 16TH FLOOR SAN FRANCISCO, CA 94104-3722 T. 415.398.5740 F. 415.398.9834	DRAWN: NJR CHECKED: DDR, RMP	SK-00

MIDDLEBROOK + LOUIE
Structural Engineers

One Bush Street
San Francisco, CA 94105
415.477.9000
Fax 415.477.9099

CLIENT :
PROJECT : 301 Mission
ENGR : CL

JOB #: 6977
DATE:
PAGE: 1 of 1

Story Information:

Level	F.F. EL	F.F. EL	H (ft)	Area (ft^2)	Perimeter (ft)
62	645 ' - 0 "	645.00	17.25	13647	482
61	627 ' - 9 "	627.75	11.00	13647	482
60	616 ' - 9 "	616.75	11.75	13647	482
59	605 ' - 0 "	605.00	12.50	13647	482
58	592 ' - 6 "	592.50	12.50	13647	482
57	580 ' - 0 "	580.00	10.75	13647	482
56	569 ' - 3 "	569.25	10.75	13647	482
55	558 ' - 6 "	558.50	10.75	13647	482
54	547 ' - 9 "	547.75	10.75	13647	482
53	537 ' - 0 "	537.00	10.75	13647	482
52	526 ' - 3 "	526.25	12.25	13647	482
51	514 ' - 0 "	514.00	10.50	13647	482
50	503 ' - 6 "	503.50	10.50	13647	482
49	493 ' - 0 "	493.00	10.50	13647	482
48	482 ' - 6 "	482.50	10.50	13647	482
47	472 ' - 0 "	472.00	10.50	13647	482
46	461 ' - 6 "	461.50	10.50	13647	482
45	451 ' - 0 "	451.00	10.50	13647	482
44	440 ' - 6 "	440.50	10.50	13647	482
43	430 ' - 0 "	430.00	12.00	13647	482
42	418 ' - 0 "	418.00	10.50	13647	482
41	407 ' - 6 "	407.50	10.50	13647	482
40	397 ' - 0 "	397.00	10.50	13647	482
39	386 ' - 6 "	386.50	10.50	13647	482
38	376 ' - 0 "	376.00	10.50	13647	482
37	365 ' - 6 "	365.50	10.50	13647	482
36	355 ' - 0 "	355.00	10.50	13647	482
35	344 ' - 6 "	344.50	10.50	13647	482
34	334 ' - 0 "	334.00	10.50	13647	482
33	323 ' - 6 "	323.50	10.50	13647	482
32	313 ' - 0 "	313.00	10.50	13647	482
31	302 ' - 6 "	302.50	10.50	13647	482
30	292 ' - 0 "	292.00	10.50	13647	482
29	281 ' - 6 "	281.50	10.50	13647	482
28	271 ' - 0 "	271.00	10.50	13647	482
27	260 ' - 6 "	260.50	10.50	13647	482
26	250 ' - 0 "	250.00	10.50	13647	482
25	239 ' - 6 "	239.50	11.25	13647	482
24	228 ' - 3 "	228.25	9.50	13647	482

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CLIENT :
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ENGR : CL

JOB #: 6977
DATE:

Story Information:

Level	F.F. EL	F.F. EL	H (ft)	Area (ft^2)	Perimeter (ft)
23	218 ' - 9 "	218.75	9.50	13647	482
22	209 ' - 3 "	209.25	9.50	13647	482
21	199 ' - 9 "	199.75	9.50	13647	482
20	190 ' - 3 "	190.25	9.50	13647	482
19	180 ' - 9 "	180.75	9.50	13647	482
18	171 ' - 3 "	171.25	9.50	13647	482
17	161 ' - 9 "	161.75	9.50	13647	482
16	152 ' - 3 "	152.25	9.50	13647	482
15	142 ' - 9 "	142.75	9.50	13647	482
14	133 ' - 3 "	133.25	9.50	13647	482
13	123 ' - 9 "	123.75	9.50	13647	482
12	114 ' - 3 "	114.25	9.50	13647	482
11	104 ' - 9 "	104.75	9.50	13647	482
10	95 ' - 3 "	95.25	9.50	13647	482
9	85 ' - 9 "	85.75	9.50	13647	482
8	76 ' - 3 "	76.25	9.50	13647	482
7	66 ' - 9 "	66.75	9.50	13647	482
6	57 ' - 3 "	57.25	9.50	13647	482
5	47 ' - 9 "	47.75	9.50	13647	482
4	38 ' - 3 "	38.25	9.50	13647	482
3	28 ' - 9 "	28.75	12.17	13647	482
2	16 ' - 7 "	16.58	16.58	13647	482
1	0 ' - 0 "	0.00	15.75	13647	482
B1	-15 ' - 9 "	-15.75	0.00	13647	482

301 MISSION STREET, SAN FRANCISCO

PEER REVIEW

1. The L-shaped columns will be in torsion for frame action along axis 2 and axis 11. Consider torsion for design.

2. The L-shaped columns support outriggers of the prime lateral system. It should be shown that participation or failure of the more rigid element will not impair the vertical and lateral resisting ability of the gravity load and lateral moment resisting system. (See section 1633.2.4.1)

3. Low-rise mat show 69 psf reinforce for total area. It looks excessive. (It is #11 @ 4.3" E.W. T & B for 8' mat.)

4. At one side of shear wall at line D.5, a ramp that has an opening in the diaphragm. Clarify how the shear will travel to both basement walls at A.1 and K.

5. Verify by calculations that ground floor diaphragm behave as a rigid diaphragm transferring forces to the perimeter basement walls and to the core. Possible reverse shear might happen in the basement and in the core walls below.

6. The mid-rise and the high-rise towers are joined at the ground floor and B1 levels. The high-rise tower has mat with piles more rigid than mid-rise 5-story basement. Verify deformation compatibility and amount of base shear that will be resisted by piles.

7. There are shear walls surrounded by openings at both sides. Verify collectors requirements to deliver shear to these walls.

8. Settlement compatibility between high-rise on piles and mid-rise on mat footing total settlement for both could be different, but there is ground floor slab without a joint that could get cracked.

9. At 9' deep mat on piles, how is the modulus of subgrade reaction applied to pile footings.

10. Is 9' deep pile cap required in full building area? There are areas where depth could be greatly reduced. (K-H for example)

11. The differential shortening in columns and walls will produce additional significant moments on outrigger beams. Is there a mechanism to relieve them from these forces?

12. Optimize P/T slab thickness at all locations.

13. Main tower moment frames are all single bay frames that are not effective. Some of the bays can't be considered as a frame because clear span to depth is less than 4 – for example B0403.

14. Please provide design criteria for outrigger beams. Are they designed as a "deep beam" with a consideration for non-linear strain distribution. What forces will be considered for designing columns that get forces from outriggers?

15. a. Columns transfer at 2nd floor line H with sloped column at 1st floor will create additional lateral component on both levels that will require beams and slab between frames to be designed for additional axial force.

    b. Very deep column section – 26' deep will act as a shear wall and attract a lot of additional seismic load to this frame. Careful considerations should be taken to design this transfer column for all applied loads.

    c. Sloped column should be included in the building model.

16. There are 4 or 5 different round column sizes on one level – ground level mid-rise. Please verify if unification of sizes is possible to reduce cost.

17. a. Design criteria on drawings describes dual system, shear wall with SMRF, and R = 8.5. Mid-rise building has no SMRF. This building also has vertical structural irregularities such as discontinuous shear wall that should be considered.

    b. Code equations 30-6 and 30-7 need not be considered for drift check.

    c. Drift check should include accidental torsion.

18. Please specify wind load design criteria for strength and for drift. Compare wind load and seismic.

19. All outriggers are unusual in shape and can't be clearly designed as a deep beams or discontinuous shear walls. Based on their importance for overall stability of the building non-linear time history analysis should be performed to investigate performance of these important elements and bring factor of safety for them to a desirable level.

301 MISSION STREET, SAN FRANCISCO

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From:	HARDIP PANNU
To:	Jack P. Moehle
Date:	1/10/2007 5:07 PM
Subject:	Re: 301 mission final letter on column test criteria ...
Attachments:	ML8821.pdf

I have attached the revised letter.
Hardip

>>> "Jack P. Moehle" <moehle@berkeley.edu> 1/10/07 4:27 PM >>>
Hardip

I trust you have been following the exchange regarding testing. We can discuss if you like on the phone.

DBI wants us to get a letter regarding the acceptance of the column tests. The letter you have drafted is fine, and I am willing to put my signature to it, but it has "draft" on the top so it is not appropriate to sign in this format. So you will need to redraft this removing the word "draft."

In discussions with Derrick, he indicates a willingness/interest to run the tests to failure, provided he has an assurance that observations beyond 0.71% strain will not be used as a basis for denying acceptance of the Baugrids in columns and walls. One option would be to insert a sentence someone where in the letter as follows:
"The undersigned encourage that the project sponsors permit testing beyond the agreed-upon longitudinal strain limit of 0.71%, with the understanding that behavior past this limit will not be considered in deciding the acceptance of Baugrids as confinement reinforcement in columns and walls." Running the tests to failure will enable us to see what stresses the Baugrids can develop in situ, which is valuable for judging the beams.

Jack

~~~~~~~~~~~~~~~~~~~~~~~~
Jack P. Moehle
email: moehle@berkeley.edu
cell: 510-407-6124
office: 510-642-3437
~~~~~~~~~~~~~~~~~~~~~~~~

MIDDLEBROOK + LOUIE
Structural Engineers

One Bush Street
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Jason J.C. Louie, S.E.
Ronald F. Middlebrook, S.E.
Hardip S. Pannu, S.E.
Robert D. McCartney, S.E.
Jeppe Larsen, EUR ING, S.E.
Navin R. Amin, S.E.

December 16, 2005

Chris Vaughn-Hulbert
Millennium Partners
735 Market Street, 3rd floor
San Francisco, CA 94103

RE: Review of Shoring Impact on CALTRANS Building
301 Mission Street, San Francisco
M + L Job #6977

CHRIS, we have completed our review regarding the impact of the shoring and about 10 to 12 feet of excavation on 151 Fremont Street CALTRANS building. The extent of review was limited to the effect of shoring and excavation work limited to the clouded area shown on the attached sketch. Our review was based on the following drawings that were made available by Millennium Partners. The drawings were labeled as "SAN FRANCISCO OAKLAND BAY BRIDGE RECONSTRUCTION DIVISION OFFICES".

Sheet No.	Date
2	May 19, 1960
3	May 22, 1958
5	May 23, 1958
6	May 19, 1960
12	May 26, 1958
14	May 28, 1958
15	May 28, 1958
16	May 28, 1958

Based on our review of above drawings, we believe that there will be no structural effect on the building from shoring and excavation work. There may be some settlement due to vibrations that are caused when the shoring is driven into the ground. We suggest that the contractor should monitor the area in the nearby vicinity for potential settlements.

CHRIS, let us know if you have any questions.

MIDDLEBROOK + LOUIE

Hardip Singh

Hardip S. Pannu, S.E.
Principal

HSP/rhc

HPANNU@MPLUSL.COM
WWW.MPLUSL.COM

STATE OF CALIFORNIA • DEPARTMENT OF TRANSPORTATION

CERTIFICATION OF STRUCTURAL EXPERIENCE

TR-0133 (NEW 02/2004)

I, Hardip S. Pannu, a licensed Structural (STRUCTURAL / CIVIL) Engineer in the State of California, attest to, that I am / was responsible for the plan set design and preparation of calculations for the project described as City heights (PROJECT NAME), located at Pellier Park (STREET ADDRESS or DISTRICT / COUNTY / ROUTE / POST MILE), San Jose (CITY / TOWN), California.

I certify and attest to, that I have five years or more of experience in Structural (SUB-STRUCTURAL / STRUCTURAL) plan set design and preparation of calculations, to include Structural Review Approval.

STRUCTURAL REVIEW APPROVAL, SUB-STRUCTURAL REVIEW APPROVAL, TUNNELS, TUNNEL SUPPORT SYSTEMS, OR STRUCTURAL FALSEWORK

List prior projects of responsibility:

PROJECT NAME	CONTACT NUMBER
Highland Hospital, Oakland, California	(510)452-2118
Franchise Tax Board, Sacramento	(925) 558-1900
621 Capitol Mall, Sacramento	(415)356-8625

PREPARING REGISTERED ENGINEER'S STAMP

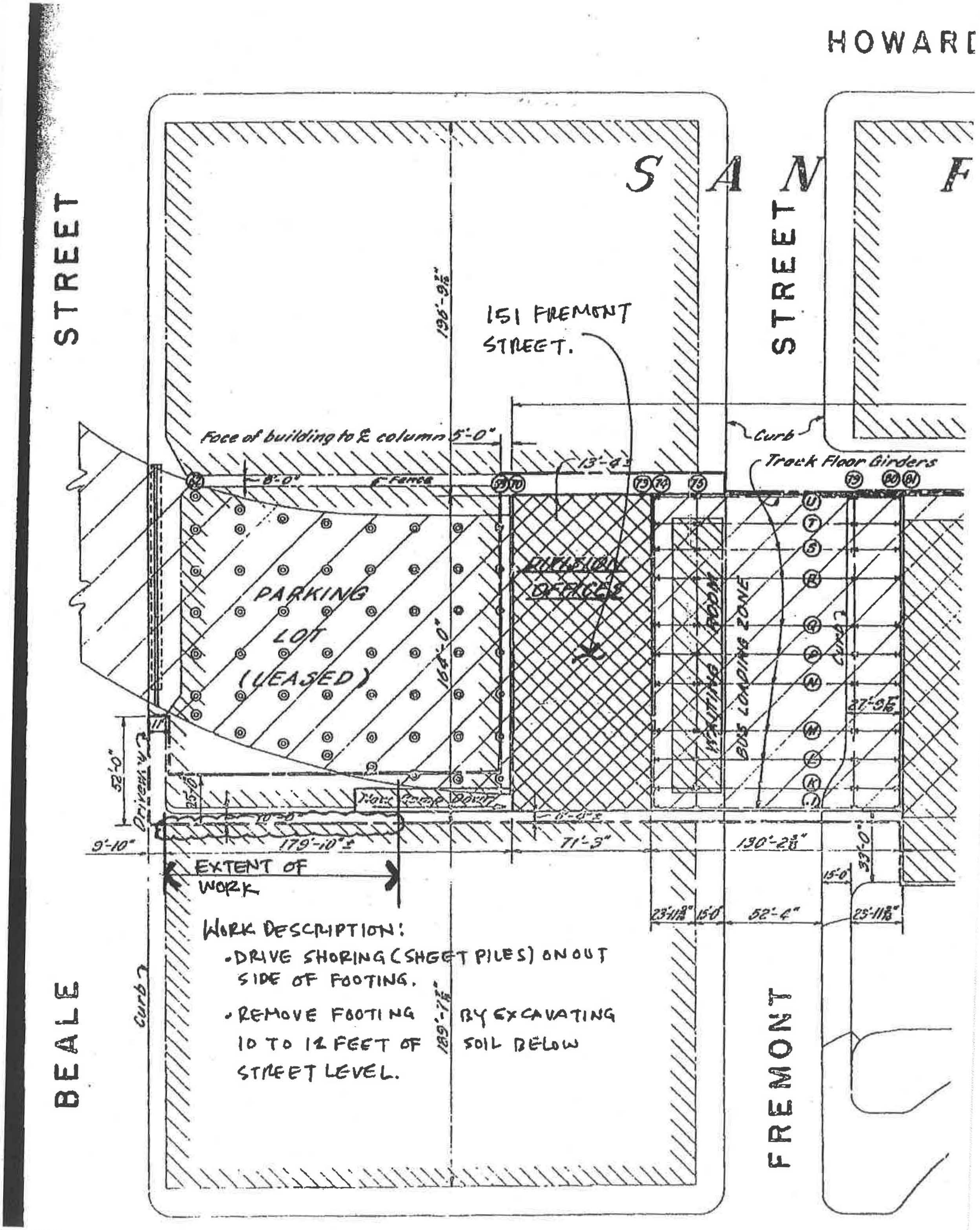
REGISTERED PROFESSIONAL ENGINEER
HARDIP S. PANNU
NO. 3168
EXP. JUNE 30, 2006
STRUCTURAL
STATE OF CALIFORNIA

I attest to the technical information contained herein and have judged the qualifications of all technical specialists providing engineering data upon which recommendations, conclusions, and decisions were based.

SIGNATURE [signature]

S-3168 REGISTERED ENGINEER

DATE 12/16/05


HOWARD
S A N F
BEALE STREET
FREMONT STREET
151 FREMONT STREET.
Face of building to ℄ column 5'-0"
Curb
Track Floor Girders
8'-0"
℄ Fence
13'-4"
PARKING LOT (LEASED)
DIVISION OFFICES
WAITING ROOM
BUS LOADING ZONE
196'-9 1/2"
164'-0"
27'-9 5/8"
52'-0"
Driveway
11'
25'-0"
9'-10"
179'-10"±
71'-3"
130'-2 3/8"
33'-0"
15'-0"
23'-11 5/8"
15'-0"
52'-4"
25'-11 5/8"
189'-7 1/2"
EXTENT OF WORK
WORK DESCRIPTION:
•DRIVE SHORING (SHEET PILES) ON OUT SIDE OF FOOTING.
•REMOVE FOOTING BY EXCAVATING 10 TO 12 FEET OF SOIL BELOW STREET LEVEL.

301 MISSION STREET, SAN FRANCISCO

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DEPARTMENT OF BUILDING INSPECTION

City & County of San Francisco
1660 Mission Street, 2nd Floor, San Francisco, California 94103-2414

December 6, 2006

Mr. Hardip Pannu
Middlebrook + Louie
One Bush Street, Suite 1300
San Francisco, CA 94104

sent via email: hpannu@mplusl.com

Subject: 301 Mission Street (Permit Application Nos. 2002/1023/9696 & 2006/0926/3344)
BauGrid® Reinforcement

Dear Jack,

Thank you for your continued work in peer reviewing the use of BauGrid reinforcement at the 301 Mission Street project. At this time, our original charge to the Structural Peer Review Panel (SPRP) regarding the review of BauGrid reinforcement has changed. Previously, the Engineer of Record (EOR), DeSimone Consulting Engineers, requested a review of the BauGrid reinforcement as a one-to-one substitution for conventional stirrups and tie reinforcement in columns, beams, and shear walls. It is apparent from the studies to date that the adequacy of the BauGrid reinforcement as a one-to-one substitution will be difficult to prove and beyond the planned scope of testing. Consequently, the Department of Building Inspection (DBI) is requesting the SPRP to continue their review with a modified charge.

DBI respectfully requests that the SPRP review the use of BauGrid reinforcement in compression dominated members (columns and shear walls) based on a performance criteria developed by the EOR, reviewed and agreed upon by the SPRP, and approved by DBI. The performance criteria shall be based on expected building performance for a Maximum Considered Earthquake including orthogonal affects and an appropriate safety factor.

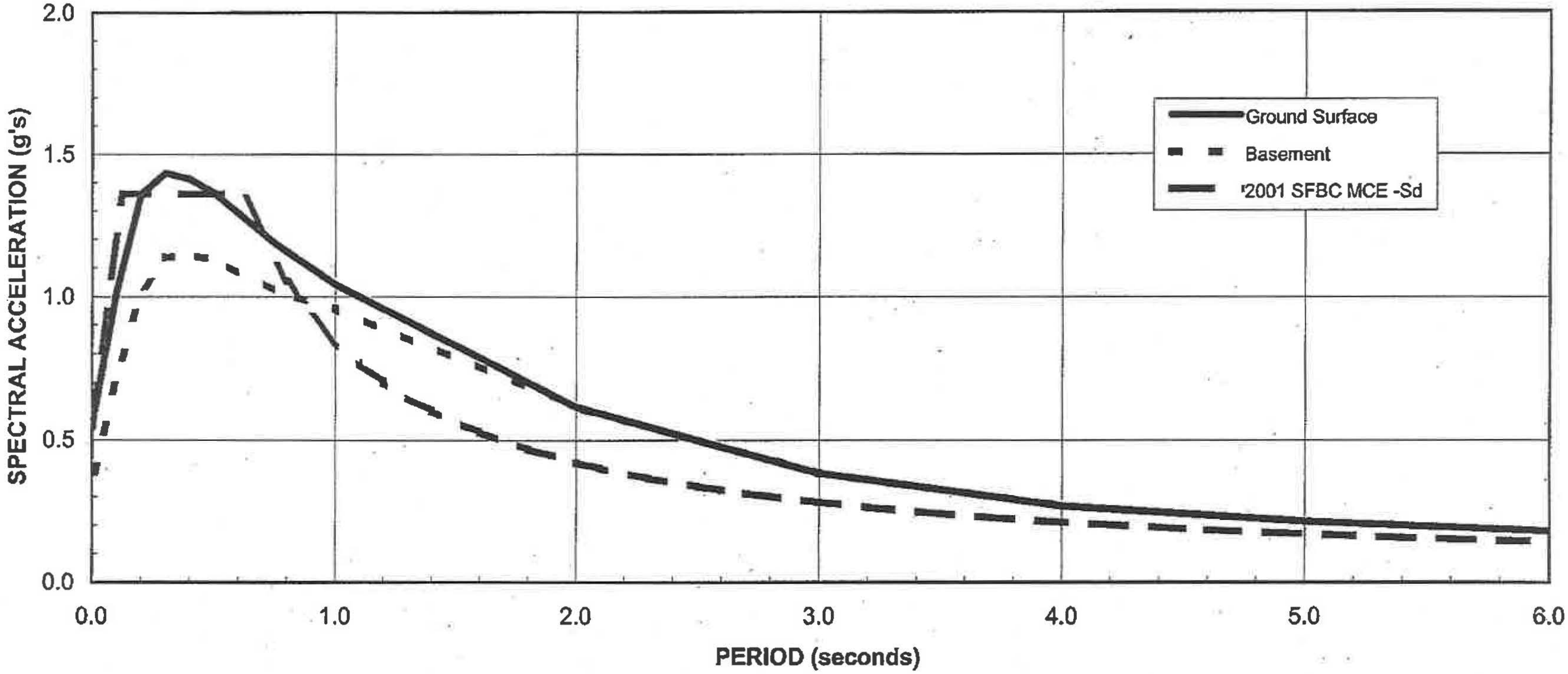
Once again, thank you for your efforts. If you have any questions or comments, please do not hesitate to call.

Sincerely,

Hanson Tom

Hanson Tom, S.E.
Principal Engineer

Cc: Amy Lee, Acting-Director
Wing Lau, Deputy Director
Carla Johnson, Acting-Deputy Director
Yan Yan Chew, Gary Ho, Howard Zee, C.S. Hwang, Raymond Lui, DBI
Dan Lowrey, Tam Chiu, DBI
Jonathan Rothstein, Senior Project Manager, Millennium Partners
Steve Hood, Project Manager, Millennium Partners
Derrick Roorda, DeSimone Consulting Engineers



Damping Ratio = 5%

Note: MCE has a 10% probability of exceedance in 100 years.

301 MISSION STREET
San Francisco, California

RECOMMENDED SPECTRA - MCE

Date 03/07/05	Project No. 3157.02	Figure D-5

Treadwell&Rollo

301 Mission Street
10 percent probability of Exceedance in 100 years (MCE)
Spectral Acceleration (g's)
Damping Ratio = 5 percent

Period (seconds)	Ground Surface	Basement
0.01	0.600	0.385
0.1	1.015	0.711
0.2	1.360	1.019
0.3	1.435	1.136
0.4	1.413	1.144
0.5	1.364	1.129
0.75	1.187	1.019
1.0	1.044	0.958
2.0	0.616	0.616
3.0	0.384	0.384
4.0	0.268	0.268
5.0	0.214	0.214
6.0	0.179	0.179

PSHA based on Working Group 2002 Seismic Hazard Model

Note: We recommend the basement spectrum be used at the foundation level for design.

PRELIMINARY REPORT
WIND-INDUCED STRUCTURAL RESPONSES
301 MISSION STREET
SAN FRANCISCO, CALIFORNIA

Project #04-1633
August 20, 2004

Prepared By:
Rowan Williams Davies & Irwin Inc.
650 Woodlawn Road West, Guelph, Ontario, Canada N1K 1B8

Matthew T. L. Browne, P.Eng., Senior Engineer
Jonathan B. Lankin, P.Eng., Project Manager

Wind tunnel tests to determine the wind-induced structural responses for the proposed 301 Mission Street tower in San Francisco, California, have been completed. This report provides the preliminary results. The objectives of this study were (i) to provide data on the wind-induced forces and moments for the structural design of the tower, and (ii) to determine the wind-induced accelerations at the top occupied floor of the tower.

The model study was carried out using the high-frequency force-balance technique. The tests were conducted on a 1:400 scale model of the building in the presence of all surroundings within a full-scale radius of 1600 ft in RWDI's boundary-layer wind tunnel. Beyond the modelled area, the upwind terrain was simulated appropriately for each wind direction. The tests were conducted for the following three configurations of surroundings:

Configuration 1: 301 Mission Street development in place with all existing surrounding buildings.

Configuration 2: 301 Mission Street development in place with all existing and future (Transbay redevelopment) surrounding buildings, with the Transbay Tower at 550 ft.

Configuration 3: 301 Mission Street development in place with all existing and future (Transbay redevelopment) surrounding buildings, with the Transbay Tower at 800 ft.

Details of testing and analysis methods will be provided in the final report. The figures and tables in this preliminary report are numbered as they will appear in the final report.

The results have been analysed including the effects of the directionality in the San Francisco wind climate. The statistical wind climate model used to determine the predicted peaks was based on local surface wind measurements taken at San Francisco International Airport. This statistical model of the local wind climate accounts for the variability of extreme wind speed with wind direction. The wind climate model was scaled so that the magnitude of the wind velocity for a 50-year return period corresponds to a fastest-mile wind speed of 70 mph at 33 ft above ground in open terrain. This speed corresponds to the value identified for the San Francisco area in the 1998 California Building Code.

Wind-Induced Forces and Moments

The overall wind-induced overturning moments, shear forces, and torsional moments acting on the 301 Mission Street tower at the "BASE" level (at grade) have been predicted for a return period of 50 years and are presented in Table 2 for the three test configurations. Note that the wind loads provided herein are for the overall design of the tower. Based on correspondence with the structural engineer, the loading provided considers only the wind loads acting on the footprint of the tower extending down to grade through the atrium (low-rise structure attached to the tower on the east side). Therefore, the loads acting on the rest of the development, outside the tower footprint, are not included in the results presented in this report. The coordinate system and reference axis used to define the forces and moments is illustrated in Figure 2. The loads were determined using the fundamental building vibration frequencies, listed in Table 2, and the corresponding mode shapes, as provided by DeSimone Consulting Engineers, PLLC on July 22, 2004. The wind-induced loads were determined for a damping ratio of 2% of critical, which was specified by the structural engineer.

Note that the wind loads provided in this report include the effects of the directionality in the local wind climate. These loads do not contain safety or load factors and are to be applied to the building's structural system in the same manner as would wind loads calculated by code analytical methods.

Effective static wind loads that correspond to the predicted overall moments and shears are provided on a floor-by-floor basis in Table 3. These loads represent the worst-case results from the three test configurations. The load distributions were determined by considering the effects of both the mean and dynamic wind loads for representative wind directions producing high loads in each of the x, y, and z (torsional) directions.

In using the predicted wind loads from Table 3, it is important to consider how the x, y, and z (torsional) components of the wind load should be combined when applying them to the structure. A set of recommended load combinations are provided in Table 4. There are basically 24 combinations in the table which represent each of eight possible sign sets (+++, ++-, +-+ etc.) with each of Fx, Fy, and Mz reaching their individual maximum percentages for that sign set. As an example of applying the combination factors, let us consider Load Case 1 of Table 4. This load case requires the application of +100% of the Fx floor-by-floor loads, +60% of the Fy floor-by-floor loads, and +45% of the Mz floor-by-floor loads from Table 3. **It is recommended that all load cases be considered for overall structural design.**

Deflections

Deflections have not been specifically evaluated in this study. Normally the structural engineer evaluates floor-to-floor and overall deflections by applying the wind load distributions derived from the wind tunnel tests to a structural computer model of the building. These deflections may then be reviewed by the structural engineer to assess the potential for problems in wall systems and partitions due to excessive shearing.

Discussion of Acceleration Criteria

The accelerations discussed herein are peak values expected to occur a few times each hour during a wind storm, not root-mean-square values which are sometimes also used in discussions of building motion issues. It should be noted that acceleration levels that are acceptable to people are dependent on many physiological factors and consequently are subjective to some degree. Some background to the suggested criteria for acceptability of building accelerations is discussed below.

Research indicates that people first begin to perceive accelerations when they reach about 5 milli-g (where milli-g is 1/1000 of the acceleration of gravity). This benchmark is thus a value that one would not want occurring too frequently in a building. However, it is not realistic to require that no accelerations ever occur above this level and so criteria have been developed that relate acceleration level to various frequencies of occurrence.

The first building code document to give guidance on building motions was the National Building Code of Canada (NBCC). It suggested that 10-year return period accelerations in the range of 1.0% to 3.0% of gravity (10 to 30 milli-g) were acceptable, with the upper end of the range being appropriate for office buildings and the lower end for residential buildings. Many towers constructed during the 1980's and 1990's were wind tunnel tested. For these towers, acceleration criteria were developed based on a consensus of the design teams, the developers and the wind engineering community. The commonly used acceleration criteria were to use a 10-year limit of between 20 and 25 milli-g for office buildings and approximately 15 to 18 milli-g for residential buildings. For the 301 Mission Street tower, in view of its residential usage, a 10-year criterion of about 15 to 18 milli-g appears appropriate according to these traditional criteria.

Research conducted subsequent to the introduction of motion criteria in the NBCC indicates that peoples' sensitivity to motion becomes less as the natural frequency of the building becomes lower (at least in the range of interest for tall buildings, 0.1 Hz to 1.0 Hz). This dependence is not reflected in the NBCC which provides a single set of criteria based on results for frequencies primarily in the range 0.15 to 0.3 Hz. The criteria suggested by the International Organization for Standardization (ISO) do include a frequency dependence and set limits where approximately 2% of those occupying the upper third of a building may object to its motions. Also the ISO criteria generally use a shorter return period than 10 years (i.e., 1 and 5 years). RWDI estimates the corresponding 10-year criterion to be about 1.2 times the 5-year criterion. For residential buildings it may be desirable to be somewhat lower than the ISO criteria.

Acceleration Predictions and Acceptability

The predicted wind-induced accelerations at the top occupied floor of the 301 Mission Street tower, taken as the "60" level (592.50 ft above the "BASE" level), are summarized in Figure 6. These accelerations represent the worst-case results from the three test configurations. Figure 6 also presents various acceleration criteria as described above. The peak total accelerations were determined as a function of return period for the provided building masses, frequencies, and an overall damping ratio of 2% of critical. The torsional acceleration component was calculated at a representative distance (47.9 ft), equal to the mass radius of gyration of the upper floors, from the central axis of the tower (given in Figure 2).

From Figure 6, it can be seen that the predicted peak accelerations are within the ISO based criteria for the 1, 5, and 10-year return periods. The 10-year accelerations are also within the commonly used criteria of 15 to 18 milli-g for a residential tower. Therefore, it is our opinion that the predicted accelerations are acceptable for human comfort in a residential building. It should be noted that building accelerations are a serviceability issue and typically not a safety issue provided the associated deflections are accounted for in the structural design and the cladding/glazing system design.

Should you have any comments or questions, or wish us to re-analyse the results for different structural properties (i.e., frequencies, damping or floor masses), please contact us.

TABLES

Table 2: Summary of Predicted 50-Year Return Period Peak Wind-Induced Overall Structural Loads on Tower at the Base Level

Configuration	Moments			Shears	
	My (lb-ft)	Mx (lb-ft)	Mz (lb-ft)	Fx (lb)	Fy (lb)
1	7.31e+08	5.00e+08	3.84e+07	2.00e+06	1.30e+06
2	7.64e+08	5.14e+08	5.25e+07	1.96e+06	1.32e+06
3	7.67e+08	5.22e+08	5.49e+07	1.95e+06	1.34e+06

Notes: (1) The above loads are the cumulative summation of the wind-induced loads at the "BASE" level (at grade) centered about the reference axis shown in Figure 2, exclusive of combination factors.

(2) A total damping ratio of 2.0% of critical was used for structural load calculations.

(3) The above loads are based on the structural properties provided by DeSimone Consulting Engineers, PLLC on July 22, 2004. The natural building frequencies were as follows:

Mode 1: 0.226 Hz (primarily X)
Mode 2: 0.230 Hz (primarily Y)
Mode 3: 0.236 Hz (primarily torsion).

Table 3: 50-Year Return Period Effective Static Floor-by-Floor Wind Loads Acting on Tower - Worst-Case Results

Floor Level	Height above Base Level (ft)	Fx (lb)	Fy (lb)	Mz (lb-ft)
BASE	0.00	9700	7800	75000
2	15.00	18500	14500	183000
3	28.00	15400	11800	187000
4	37.75	15600	10100	205000
5	47.50	15600	10100	226000
6	57.25	15600	10100	247000
7	67.00	15600	10100	266000
8	76.75	15500	10100	283000
9	86.50	15500	10100	300000
10	96.25	15500	10100	314000
11	106.00	15400	10100	327000
12	115.75	15400	10100	340000
13	125.50	15900	10100	359000
14	135.25	17800	10900	401000
15	146.63	18700	11200	431000
16	156.38	18100	11000	439000
17	166.13	18500	11400	452000
18	175.88	19000	11900	471000
19	185.63	19700	12500	497000
20	195.38	20400	13000	521000
21	205.13	21200	13600	545000
22	214.88	21900	14200	572000
23	224.63	22800	14800	599000
24	234.38	23600	15400	627000
25	244.13	24400	16000	654000
26	253.88	25200	16600	682000
27	263.63	26100	17200	710000
28	273.38	26900	17800	738000
29	283.13	27800	18400	766000
30	292.88	28600	19000	794000
31	302.63	29500	19600	823000
32	312.38	30400	20300	852000
33	322.13	33400	22300	933000
34	333.50	34200	23700	1004000
35	343.25	33200	23000	997000
36	353.00	33300	23100	993000
37	362.75	33800	23400	1004000
38	372.50	34700	24000	1033000
39	382.25	35600	24600	1063000
40	392.00	36500	25200	1094000
41	401.75	37200	25700	1110000

Table 3: 50-Year Return Period Effective Static Floor-by-Floor Wind Loads Acting on Tower - Worst-Case Results

Floor Level	Height above Base Level (ft)	Fx (lb)	Fy (lb)	Mz (lb-ft)
42	411.50	38100	26300	1140000
43	421.25	39100	26900	1170000
44	431.00	40000	27400	1200000
45	440.75	40900	28000	1231000
46	450.50	41900	28600	1261000
47	460.25	42800	29100	1292000
48	470.00	43700	29700	1323000
49	479.75	44600	30200	1354000
50	489.50	45500	30800	1385000
51	499.25	46400	31300	1416000
52	509.00	48100	32300	1476000
53	518.75	50400	33600	1566000
54	528.50	53200	35600	1637000
55	539.88	54300	36400	1646000
56	550.29	53400	35600	1676000
57	560.71	54300	36200	1712000
58	571.13	55300	36700	1749000
59	581.54	56500	37600	1788000
60	592.50	60300	40400	1850000
ROOF	605.00	84800	57100	2363000
UPPER ROOF	627.00	50500	36900	594000
Total		2.00e+06	1.34e+06	5.49e+07

Notes: (1) The loads given in this table should be used with the load combination factors given in Table 4.

(2) The loads given in this table are centered about the reference axis shown in Figure 2.

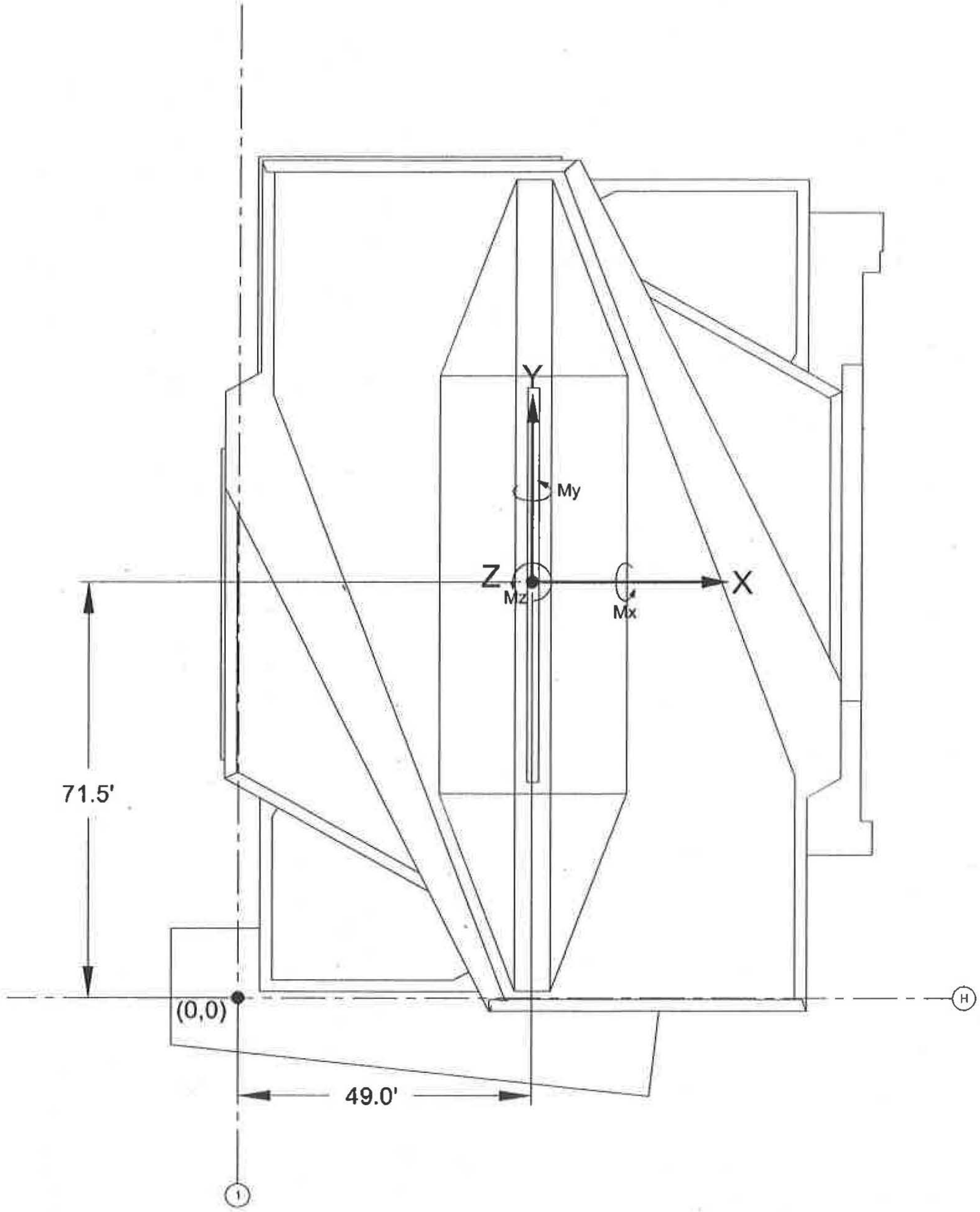
(3) The loading provided considers only the wind loads acting on the footprint of the tower extending down to grade through the atrium.

Table 4: Recommended Load Combinations for Simultaneous Application of Effective Static Floor-by-Floor Loads from Table 3

Load Combination	Recommended Load Combination Factors of 50-Year Return Period Wind Loads		
	X Forces (F_x)	Y Forces (F_y)	Torsional Moment (M_z)
1	+100%	+60%	+45%
2	+100%	+60%	-30%
3	+100%	-30%	+45%
4	+100%	-30%	-30%
5	-90%	+35%	+30%
6	-90%	+35%	-30%
7	-90%	-40%	+30%
8	-90%	-40%	-30%
9	+55%	+100%	+45%
10	+55%	+100%	-30%
11	+30%	-85%	+30%
12	+30%	-85%	-30%
13	-40%	+100%	+45%
14	-40%	+100%	-30%
15	-50%	-85%	+30%
16	-50%	-85%	-30%
17	+55%	+60%	+100%
18	+55%	+60%	-90%
19	+55%	-30%	+100%
20	+55%	-30%	-90%
21	-30%	+60%	+100%
22	-30%	+60%	-90%
23	-30%	-30%	+100%
24	-30%	-30%	-90%

Note: (1) Load combination factors have been produced through consideration of the structure's response to various wind directions, modal coupling, correlation of wind gusts and the directionality of strong winds in the local wind climate.

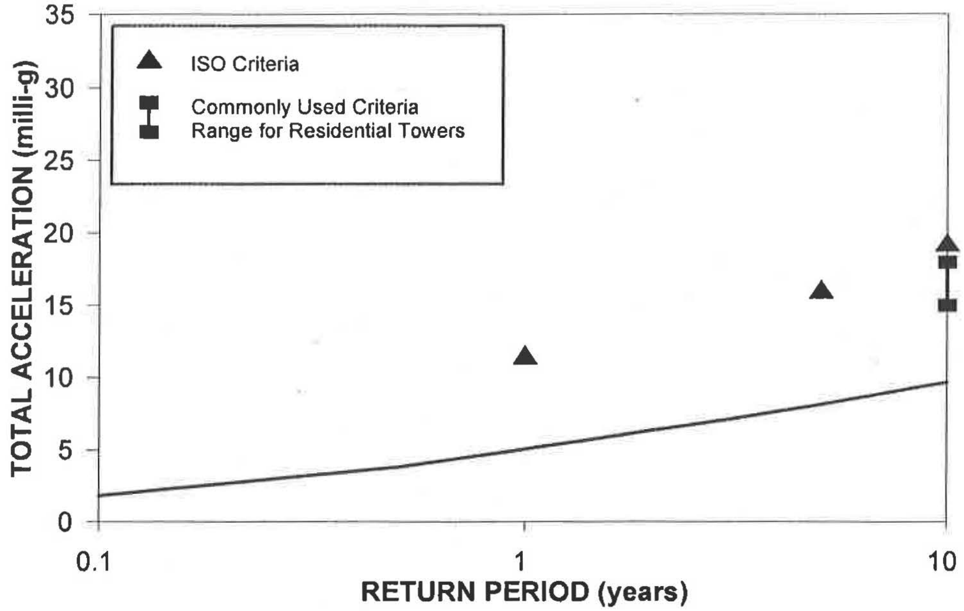
FIGURES



Note:
Point (0,0) indicates co-ordinate origin provided by the structural engineer.

0 15 30ft

Co-ordinate System for Structural Loading	True North	Drawn by: DJM	Figure: 2	RWDI
		Approx. Scale:	1"=30'	
301 Mission Street - San Francisco, California	Project #04-1633	Date Revised:	Aug. 17, 2004	



Return Period (Years)	Peak Total Accelerations (milli-g)	ISO (3) Criteria (milli-g)
1	5.1	11.6
5	8.2	16.1
10	9.7	19.4 (4)

Notes:

(1) A damping ratio of 2% of critical was used.

(2) Accelerations are predicted at the "60" level (592.50 ft above the "BASE" level) at a radial distance of 47.9 ft from the central axis of the tower (given in Figure 2).

(3) ISO is the International Organization for Standardization, and provides acceleration criteria for buildings for the 1 and 5-year return periods.

(4) RWDI extrapolation of ISO criteria to the 10-year return period.

(5) The commonly used acceleration criteria range for a residential tower is 15 to 18 milli-g at the 10-year return period.

Predicted Peak Accelerations at Top Occupied Floor **Worst-Case Results** 301 Mission Street - San Francisco, California — Project #04-1633	Figure No. 6 Date: Aug. 20, 2004	RWDI

DESIMONE

NEW YORK
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SAN FRANCISCO
NEW HAVEN
LAS VEGAS

October 23, 2006

City and County of San Francisco
Department of Building Inspection
1660 Mission Street, 2nd Floor
San Francisco, CA 94103

DeSimone Project #4069B
301 Mission Street

Attn: Mr. Hanson Tom, S.E.
Principal Engineer

Re: Letter from H. Tom (City of SF) to D. Roorda (DeSimone), dated October 13, 2006, Re. 301 Mission Street (Permit No. 2002/1023/9696) - BauGrid® Reinforcement

Dear Hanson,

DeSimone has worked closely and collaboratively over the last week with Professor Jack Moehle from the University of California at Berkeley, and Professor Murat Saatcioglu from the University of Ottawa, with a goal of developing a test procedure to demonstrate that BauGrid reinforcement is appropriate for use with 10,000 psi concrete in conjunction with the 301 Mission Street project.

As a result of these discussions, we have agreed to test three identical concrete column specimens as depicted in the attached sketch. As you can see, this specimen differs in a number of ways from that which you described verbally in your letter of October 13. However, we believe, and both Professors Moehle and Saatcioglu agree, that this test specimen accurately reflects the actual conditions being used at 301 Mission Street, and that successful testing of this specimen will demonstrate the adequacy of BauGrid for this project.

Please note the following:

- The 15"x15" cross section is the same as you suggested.
- We propose to use a 9-cell BauGrid arrangement consisting of #4 size bars. We realize that #4 bars are smaller than the #5 BauGrids being used at the 301 Mission Street project. However, this scaling of reinforcement is necessary to provide a test column configured with similar transverse reinforcing steel ratio and confinement efficiency as the cross tie configurations used in the actual project. We will work with Prof's Moehle and Saatcioglu to develop a testing procedure for the BauGrid material in order to demonstrate equivalent performance of #5 and #4 materials. We expect that this test will be similar to those performed previously by Prof. Saatcioglu in which he demonstrated that BauGrids had sufficient ductility to act effectively as confinement reinforcing.
- We propose to use 12-#7 vertical bars. This represents a vertical steel ratio of 3.2%, which is nearly two times greater than that in the boundary elements at 301 Mission Street. Note that a 12-bar pattern is necessary for use in conjunction with the 9-cell BauGrid configuration.

- We propose to use Gr. 60 vertical bars since the grade of the vertical steel should not significantly influence the outcome of the tests. Further, we know that these bars are readily available and we do not know if additional time would be required to procure Gr. 75 #7 bars.

We have not yet concluded our discussions with Professors' Moehle and Saatcioglu regarding the specifics of the testing procedure (loading application, instrumentation, etc.) and appropriate acceptance criteria. However, we are all in agreement with the test specimen as shown in the attached sketch. If you are in agreement, we would like to proceed with fabrication of the test specimens immediately, and will continue our discussions of these related and important issues while that effort takes place.

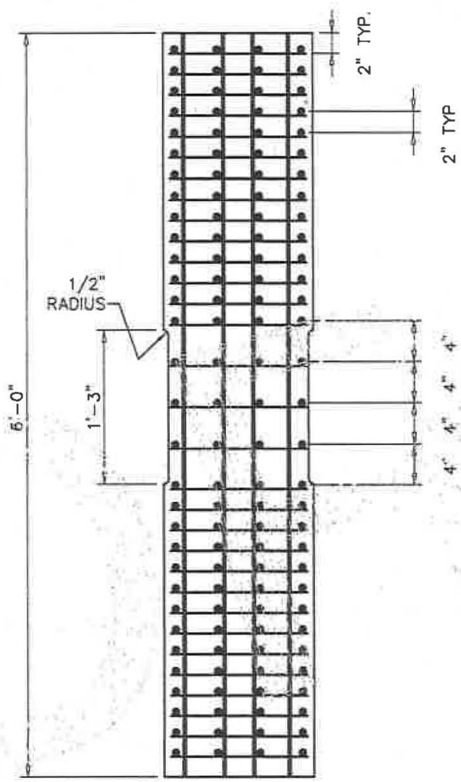
Please review the sketch and provide us with a statement indicating that testing of these specimens will be adequate to demonstrate to SFDBI that BauGrid is acceptable for use on the 301 Mission Street project. Upon receipt of this statement, we will forward this information to the project sponsor and contractors so that fabrication of the specimens can begin immediately. As you and I have discussed, the timeframe associated with fabrication of the test specimens will be controlled by the contractors. We will update you upon receipt of any and all information regarding this timeframe. Please accept our assurances that we want this test to be completed in the timeliest manner possible.

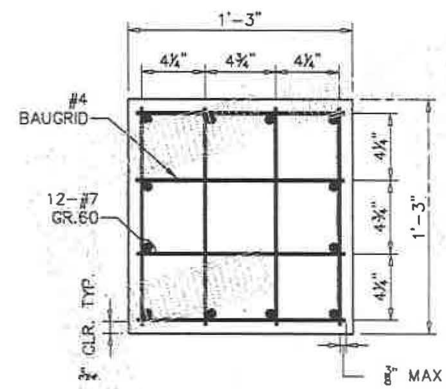
We trust that you will find the above explanation a satisfactory response to your concerns. If you have any additional concerns, please contact me directly at your earliest convenience.

DESIMONE CONSULTING ENGINEERS, PLLC

Derrick D. Roorda, SE
Senior Associate

cc: Mr. Gary Ho, City & County of SF
Jonathan Rothstein, Steven Hood (Millennium Partners)
Mr. Stephen DeSimone, Dr. Ronald Polivka, Mr. Nicolas Rodrigues (DeSimone)
Prof. Jack Moehle, U.C. Berkeley
Prof. Murat Saatcioglu, University of Ottawa
Hardip Pannu, Middlebrook + Louie



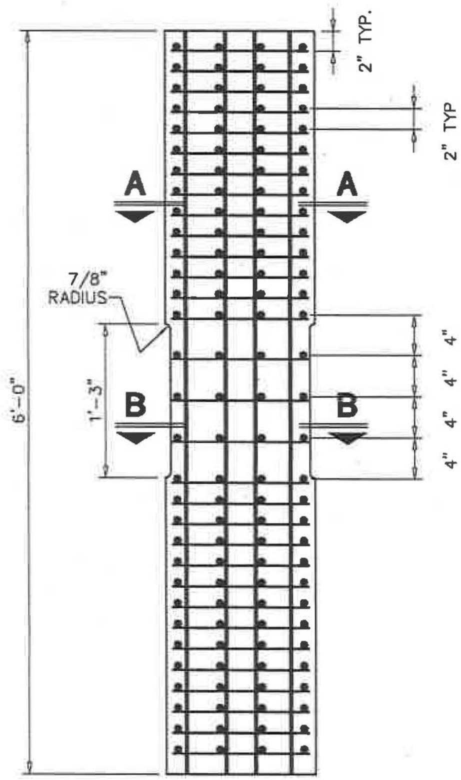


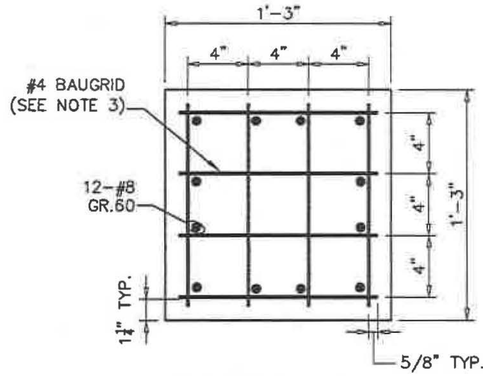
BAUGRID PLAN DETAIL

NOTES:

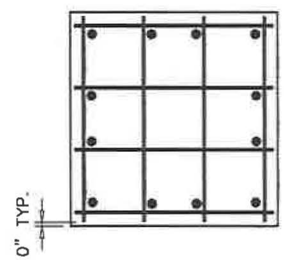
1. 31 BAUGRIDS PER SPECIMEN.

PROJECT: 301 MISSION	JOB #: 4069	SCALE: 1" = 1'-0"
TITLE: BAUGRID COLUMN TEST SETUP	DATE: 10/20/2006	DWG. NO.
DESIMONE 160 SANSOME STREET 16TH FLOOR SAN FRANCISCO, CA 94104 - 3722 T. 415.398.5740 F. 415.398.9834	DRAWN: NJR CHECKED: NJR	SK-01





SECTION A-A

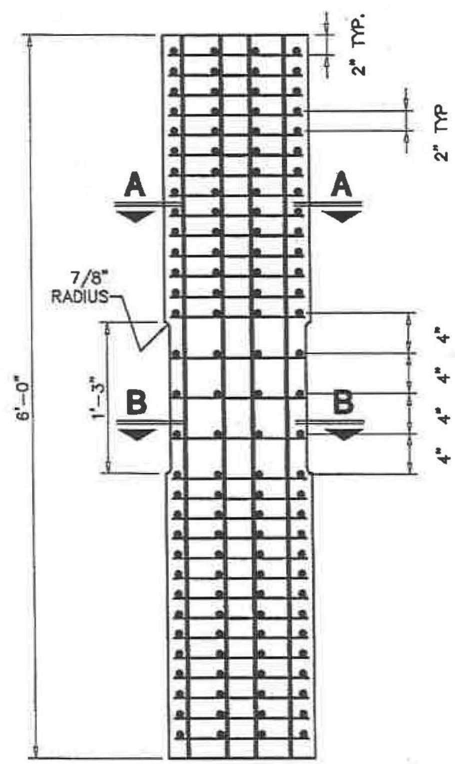


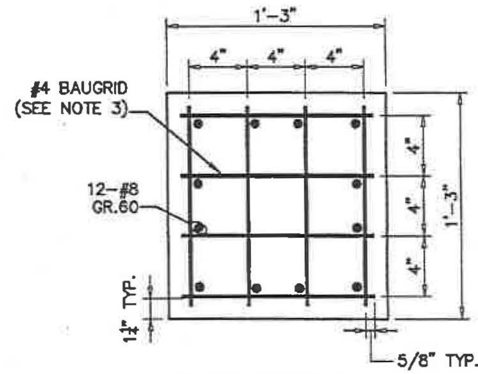
SECTION B-B

NOTES:

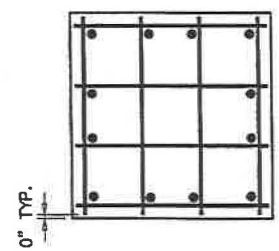
1. 31 BAUGRIDS PER SPECIMEN.
2. CONCRETE F'c = 10,000 PSI MIN.
3. THE MATERIAL SHALL BE THE SAME AS THE MATERIAL USED IN THE 301 MISSION STREET PROJECT.

PROJECT: 301 MISSION	JOB #: 4069	SCALE: 1" = 1'-0"
TITLE: BAUGRID COLUMN TEST SETUP	DATE: 10/30/2006	DWG. NO.
DESIMONE 160 SANSOME STREET 16TH FLOOR SAN FRANCISCO, CA 94104-3722 T. 415.398.5740 F. 415.398.9834	DRAWN: NJR CHECKED: DDR, RMP	SK-01





SECTION A-A



SECTION B-B

NOTES:

1. 31 BAUGRIDS PER SPECIMEN.
2. CONCRETE F'c = 10,000 PSI MIN.
3. THE MATERIAL SHALL BE THE SAME AS THE MATERIAL USED IN THE 301 MISSION STREET PROJECT.

"CITY APPROVED"

NJR
DeSimone
10/30/06 6:20 pm

PROJECT: 301 MISSION		JOB #: 4069	SCALE: 1" = 1'-0"
TITLE: BAUGRID COLUMN TEST SETUP		DATE: 10/30/2006	DWG. NO.
DESIMONE 160 SANSOME STREET SAN FRANCISCO, CA T. 415.398.5740	16TH FLOOR 94104-3722 F. 415.398.9834	DRAWN: NJR	SK-01
		CHECKED: DDR, RMP	

DESIMONE

NEW YORK
MIAMI
SAN FRANCISCO
NEW HAVEN
LAS VEGAS

November 03, 2006

City and County of San Francisco
Department of Building Inspection
1660 Mission Street, 2nd Floor
San Francisco, CA 94103

DeSimone Project #4069B
301 Mission Street

Attn: Mr. Hanson Tom, S.E.
Principal Engineer

Re: 301 Mission Street (Permit No. 2002/1023/9696) - BauGrid® Reinforcement Test Procedure, drawings SK-00 to SK-02

Dear Hanson,

DeSimone has developed a testing procedure and acceptance criteria that will demonstrate BauGrid reinforcement is appropriate for use on the 301 Mission St. project. The details are provided on the attached drawings SK-00 and SK-02 dated 11/03/2006. Drawing SK-01, which contains details as to the proposed BauGrid test column, was previously approved by DBI on 10/30/06, and is contained herein for completeness.

We would appreciate your timely review and approval of the proposed testing procedure and acceptance criteria. Please contact me directly if you have any questions or comments.

DESIMONE CONSULTING ENGINEERS, PLLC

Derrick D. Roorda, SE
Senior Associate

Enclosures (3) – Sheets SK-01, SK-02, and SK-03

cc: Mr. Gary Ho, City & County of SF
Mr Ray Liu, City & County of SF
Jonathan Rothstein, Steven Hood (Millennium Partners)
Mr. Stephen DeSimone, Dr. Ronald Polivka, Mr. Nicolas Rodrigues (DeSimone)
Prof. Jack Moehle, U.C. Berkeley
Prof. Murat Saatcioglu, University of Ottawa
Hardip Pannu, Middlebrook + Louie

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PROCEDURE:

1. TEST SPECIMENS
   a. SHAKEDOWN TEST. SPECIMENS A1, A2, & A3. THREE (3) SPECIMENS CONTAINING BAUGRID WILL BE BUILT PER SK-01.
   b. CITY TEST. SPECIMENS B1, B2, & B3. THREE (3) SPECIMENS CONTAINING BAUGRID WILL BE BUILT PER SK-01.
2. TEST INSTRUMENTATION
   a. ALL SPECIMENS WILL BE INSTRUMENTED WITH TWO (2) AXIAL STRAIN MEASUREMENT DEVICES (LVDT'S) ON THE EXTERIOR AND ON OPPOSITE SIDES OF THE SPECIMEN ACROSS THE TESTING REGION.
   b. ALL SPECIMENS WILL BE INSTRUMENTED WITH TWO (2) STRAIN GAGES ON THE LONGITUDINAL REINFORCEMENT. THESE GAGES WILL BE PLACED ON OPPOSITE SIDES OF THE SPECIMEN, NEAR THE LVDT'S, WITHIN THE TESTING REGION.
   c. ALL SPECIMENS WILL BE INSTRUMENTED WITH FOUR (4) STRAIN GAGES ON THE TRANSVERSE REINFORCEMENT. STRAIN GAGES ON THE TRANSVERSE BAUGRID REINFORCEMENT WILL BE PLACED AS CLOSE AS POSSIBLE TO THE WELDS.
3. PURPOSE OF EACH TEST
   a. SPECIMENS A1, A2, & A3 WILL BE TESTED WHEN THE CONCRETE STRENGTH HAS REACHED 8,000 PSI. THE PURPOSE OF THESE TESTS WILL BE TO MAKE SURE THE TESTING PROCEDURE IS UNDERSTOOD PRIOR TO TESTING THE CITY TEST SPECIMENS.
   b. SPECIMENS B1, B2, & B3 WILL BE TESTED WHEN THE CONCRETE STRENGTH HAS REACHED 10,000 PSI. THE OUTCOME OF THESE TESTS WILL DETERMINE IF BAUGRID IS ACCEPTABLE FOR USE ON THE 301 MISSION STREET PROJECT.
4. CONCRETE STRENGTH TESTS
   a. FORTY (40) CONCRETE CYLINDERS SHALL BE TAKEN FROM THE SAME CONCRETE USED FOR THE TEST SPECIMENS.
   b. TWO (2) CYLINDERS SHALL BE BROKEN ON THE 5TH DAY AFTER CONCRETE PLACMENT AND ON EACH DAY THEREAFTER UNTIL THE CONCRETE REACHES 10,000 PSI, WHICH IS EXPECTED AT APPROXIMATELY FOURTEEN (14) DAYS AFTER PLACEMENT.
   c. TWO (2) ADDITIONAL CYLINDERS SHALL BE BROKEN AT 28, 56, AND 90 DAYS AFTER CONCRETE PLACEMENT.
5. TESTING PROCEDURE
   a. EACH SPECIMEN WILL BE SUBJECTED TO MONOTONIC CONCENTRIC LOADING. (THE APPROPRIATE RATE OF LOADING IS TO BE DETERMINED AND AGREED TO PRIOR TO TESTING.)
   b. SPECIMENS A1, A2, AND A3 WILL BE LOADED UNTIL FAILURE.
   c. SPECIMENS B1, B2, AND B3 WILL BE LOADED UNTIL THEY HAVE REACHED THE ACCEPTANCE CRITERIA ONLY. ADDITIONAL LOADING MAY BE APPLIED AT THE OWNER'S SOLE DISCRETION.
6. TEST ACCEPTABILITY CRITERIA
   a. CITY TEST SPECIMENS B1, B2, & B3. EACH TEST WILL BE DEEMED SUCCESSFUL IF THE AVERAGE OF THE TWO AXIAL STRAIN DEVICES REACHES 0.6%.
7. BAUGRID EQUIVALENCY TESTS
   a. BAUGRID QUALITY CONTROL TESTS 1-5 AS SHOWN IN SK-02 WILL BE PERFORMED ON THE #4 BAUGRIDS USED IN THE TEST SPECIMENS, AS WELL AS REPRESENTATIVE #5 BAUGRIDS TO THOSE BEING USED AT THE 301 MISSION STREET PROJECT.
   b. SUCCESSFUL COMPLETION OF THESE TESTS WILL BE DEMONSTRATED IF THE #4 BAUGRIDS AND #5 BAUGRIDS ALL PASS THE ASSOCIATED ASTM AND BAUGRID QUALITY CONTROL MANUAL CRITERIA.

PROJECT: 301 MISSION	JOB #: 4069	SCALE: N.T.S.
TITLE: BAUGRID COLUMN TEST SETUP PROCUDURES	DATE: 11/03/2006	DWG. NO.
DESIMONE 160 SANSOME STREET 16TH FLOOR SAN FRANCISCO, CA 94104-3722 T. 415.398.5740 F. 415.398.9834	DRAWN: NJR CHECKED: DDR, RMP	SK-00

NOTES:

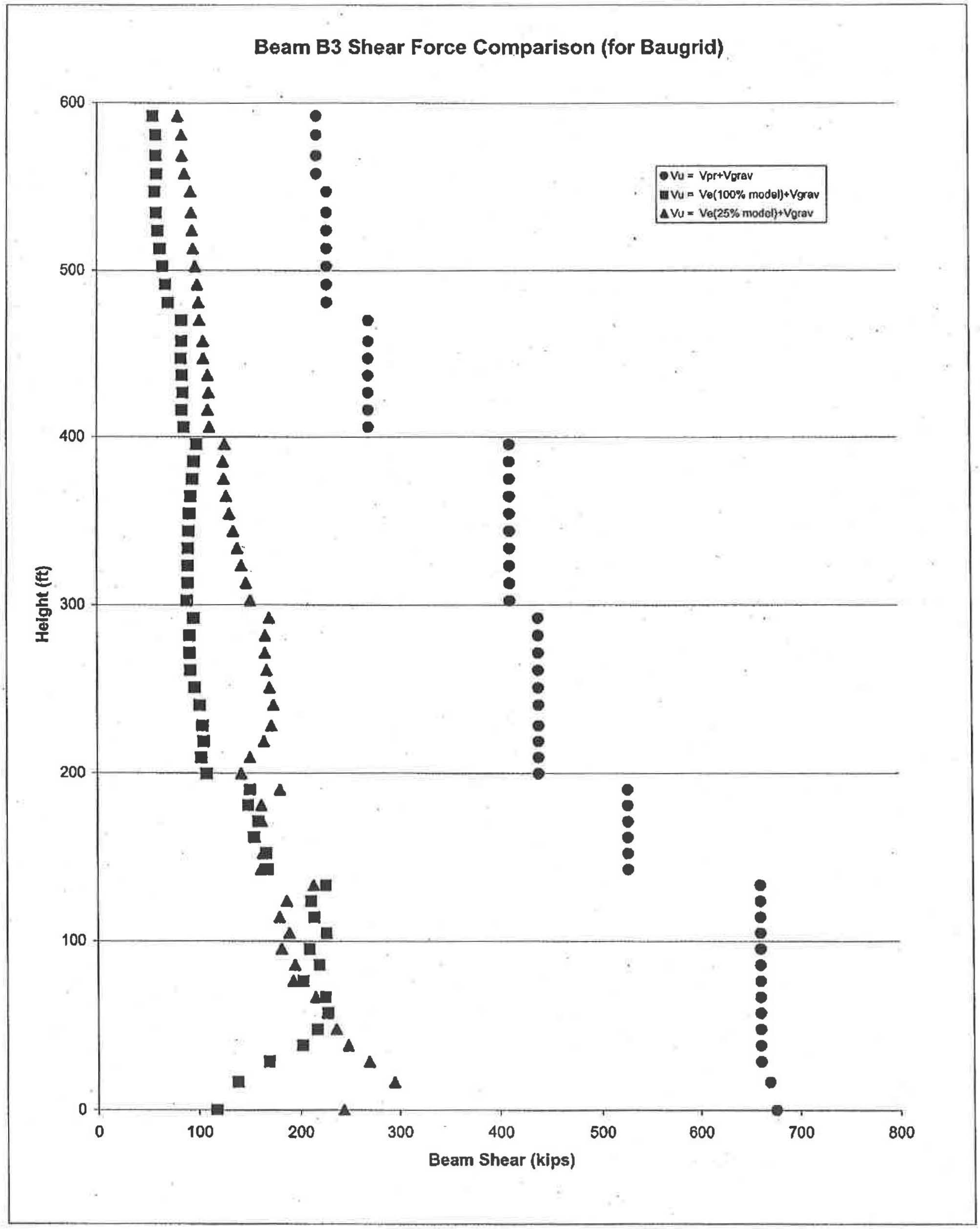
1. TESTS SHALL BE PERFORMED FOR BOTH #4 AND #5 BAUGRIDS AS SHOWN ON PAGE 29 OF THE BAUGRID QUALITY CONTROL MANUAL (REPRODUCED HERE FOR CLARITY.) ALL APPLICABLE ASTM PROCEDURES AND/OR THE BAUGRID QUALITY CONTROL MANUAL SHALL BE ADHERED TO.

2. COLUMN TEST BAUGRID FABRICATION SHOULD ONLY PROCEED IF BAUGRID COUPONS TESTS ARE SATISFACTORY.

3. TESTS SHALL BE PERFORMED IN THE MANUFACTURING FACILITY IN CHINA USING THE SAME TESTING APPARTUS AS USED FOR THE 301 MISSION PROJECT.

4. ALL TESTS SHALL BE PERFORMED WITH SMITH-EMERY AS WITNESS. SMITH-EMERY SHALL PRODUCE A REPORT PRESENTING ALL TESTING RESULTS AND A STATEMENT AS TO THE TESTS CONFORMANCE WITH THE BAUGRID QUALITY CONTROL MANUAL. ALL RAW STRESS-STRAIN DATA SHALL ALSO BE INCLUDED IN THE REPORT.

PROJECT: 301 MISSION	JOB #: 4069	SCALE: N.T.S.
TITLE: BAUGRID COUPON TEST SETUP	DATE: 11/03/2006	DWG. NO.
DESIMONE 160 SANSOME STREET 16TH FLOOR SAN FRANCISCO, CA 94104 - 3722 T. 415.398.5740 F. 415.398.9834	DRAWN: NJR CHECKED: DDR, RMP	SK-02


Beam B3 Shear Force Comparison (for Baugrid)
Vu = Vpr+Vgrav
Vu = Ve(100% model)+Vgrav
Vu = Ve(25% model)+Vgrav
Height (ft)
Beam Shear (kips)
0
100
200
300
400
500
600
700
800

The 301 Mission Street project consists of two separate structures located on the same site. The western structure (Tower) is a 58-story, 605-foot tall building over one sub-grade level. The eastern structure (Mid-rise) is a 12-story, 128-foot tall building over five sub-grade levels. The two structures are separated by a seismic joint at the B1, Ground, 2nd, and 3rd Floors.

Gravity System Description

Both structures are of cast-in-place concrete construction. The floor slabs above ground level in both structures will utilize post-tensioning, whereas the lower slabs utilize only mild steel reinforcing.

Lateral System Description

Tower. The Tower relies on a dual lateral system comprised of concrete shear walls with outriggers, and concrete special moment-resisting frames. Lateral forces from the Tower are transmitted by the core walls and the columns all the way to the pile cap at B1. The ground floor slab is not required to transfer forces to the perimeter basement walls.

Mid-rise. The Mid-rise relies solely on a concrete shear wall system. The core walls of the Mid-rise, unlike those of the Tower, have the shear shifted to the perimeter basement walls through the ground floor and basement level diaphragms.

Materials

Concrete. Concrete strengths in the Tower walls and frames vary between 7 and 10 ksi, and in the Mid-rise between 7 and 8 ksi. All floor slabs are 5 ksi.

Reinforcing. The shear walls in both buildings and the moment frames in the Tower use Grade 75 reinforcing for bars larger than #8's per the General Notes sheet.

BauGrid Welded Reinforcement Grids (WRG) manufactured by BauTech, Inc. will be used in lieu of conventional reinforcing in the Tower for ties in the walls and columns, and stirrups in beams. While the Baugrid product has ICBO approval (ER-5192), the City of San Francisco's Department of Building Inspection believed that the ICBO approval was not sufficient and that the substituted WRG may not meet various prescriptive code requirements. By utilizing section 104.2.8 of the code, the alternative materials section, DeSimone subsequently demonstrated that the substituted WRG met the same performance goals that the code implies are to be provided by conventional reinforcing.

For walls and columns, calculations were provided demonstrating the maximum demand required by a 4/3 MCE event, and a laboratory testing program was completed which showed that the WRG provided a capacity that met the demand.

For beams, calculations were provided demonstrating that the shear demand required by code is resisted by beam shear capacity with contributions from both concrete and the WRG. Capacity of the concrete in shear is based on published research. Capacity of the WRG is based on relevant testing data obtained through BauTech's QC/QA program on WRG material to be used on this project.

Foundations

Tower. The Tower foundation consists of a 10-foot thick pile cap supported by pre-cast concrete piles. The bottom of the pile cap is approximately 25' below the existing grade. The initial vertical pile displacement due to slippage required to fully engage the pile is expected to be approximately 1" by the time of project construction completion. Additional long-term pile settlement due to compression of the underlying clay layers is expected to be as much as 5". As the piles are only located directly below the Tower footprint, this settlement is expected to occur uniformly over the Tower foundation area.

Mid-rise. The Mid-rise structure rests on a mat foundation that varies between 6 feet and 8 feet in thickness. The bottom of this excavation is approximately 63 feet below the existing grade. Tie-downs resist hydrostatic uplift pressures under the portion of the deep excavation that is not directly below the Mid-rise, i.e., the area between the Mid-rise and the Tower.

Building Separation

The foundations and lateral systems of the two buildings are considered completely separate because a joint is located between them at the B1, Ground, 2nd, and 3rd Floors. "Hinge slabs" allow circulation between the two buildings, while still accommodating differential settlement and seismic displacements between the two structures.

Wind Loads

A wind tunnel study was performed and a report issued by Rowan Williams Davies & Irwin Inc. (RWDI). The results of the report were used to evaluate both the Tower and Mid-rise. Wind does not control either design forces or interstory drifts for either structure.

Seismic Loads

Site-specific ground motions provided by the geotechnical engineer of record, Treadwell and Rollo, were used for the analyses of both structures. Earthquake design forces acting on individual elements were obtained by performing response spectrum analyses with the proprietary computer program "ETABS" written by Computers and Structures, Inc. of Berkeley, California.

The following information was used to determine the seismic design forces.

Z	=	0.40Na		=	1.0
I	=	1.0	Nv	=	1.064
R	=	8.5 (Tower)	Ca	=	0.44
R	=	5.5 (Mid-rise)	Cv	=	0.67
Soil	=	Sd			

Tower. The lateral system is "regular" as defined by UBC 1629.5.2. The design forces were therefore reduced by 80% as allowed by 1631.5.4.2.

Different base shears were used for checking design forces and building interstory drifts.

Forces – Includes the building period limitation of 1.3 T_A and the minimum base shear of equation 30-6, reduced by 80% as allowed by 1631.5.4.2. (T_A is the period of the structure determined with Method A using equation 30-8.)

Drift check #1 – Per UBC. Neglecting period limitations and minimum base shears prescribed by equations 30-6 and 30-7, further reduced by 80% as allowed by 1631.5.4.2, but including the effects of torsion and of 5% mass eccentricity.

Drift check #2 – Per 2003 NEHRP provisions. This approach is widely held as the appropriate check for tall buildings with long periods and conservatively includes the equivalent of UBC equation 30-7, reduced by 80% as allowed by 1631.5.4.2. For buildings that are torsionally regular, this approach allows neglecting torsion effects for drift considerations, accomplished by evaluating drifts at diaphragm center of mass.

Mid-rise. Due to the eccentricity of the shear walls relative to the center of mass of the building, the Mid-rise exhibits a slight torsional irregularity. For this reason the base shear was not reduced in accordance with 1631.5.4.2.

Different base shears were used for checking design forces and building interstory drifts. (Since the period of the structure is relatively short, the minimum base shear equations of 30-6 and 30-7 do not apply.)

Design Procedures

All elements of the structure are designed and detailed in accordance with the load combinations and requirements of the 2001 SFBC. Additional procedures were also followed as listed below.

Steel Link Beams. The 2001 SFBC does not address the steel link beams used within the core of the Tower. These elements are designed using the 2002 AISC Seismic Provisions requirements for Special Reinforced Concrete Shear Walls Composite with Structural Steel Elements.

Capacity Design. Each of the 12 outriggers connecting to the central shear core of the Tower contains two diagonally reinforced link beam elements. These links are designed to remain elastic under the code-prescribed seismic loads, but it is desirable for them to yield first once the design loads are exceeded by a major earthquake. In order to insure this behavior, the capacities of the link beams were calculated and increased by an overstrength factor. The resulting forces were used as the demands for which the following elements were designed: the portion of each outrigger connecting to the core walls, the outrigger columns, and the pile cap.

Note that this approach is not required by the SFBC and represents an effort to "go beyond the code". This increases our confidence that in a large earthquake the very ductile link beam elements will yield first, and the critical connecting elements of the structure will remain essentially undamaged. The design of all elements still meets the requirements of the SFBC.

The outriggers columns are designed to remain elastic when simultaneously subjected to the capacity of all link beams, as well as all tributary gravity loads.

The pile cap under the Tower is designed to remain elastic when subjected to the capacities of the outrigger columns, as well as the expected maximum moment at the base of the shear wall core.

DESIMONE

Structural Calculations

301 Mission Street
San Francisco, CA

Shear Capacity of Moment Frame Beams
Reinforced with BauGrid

Prepared for:
San Francisco Department of Building Inspection

Prepared by:
DeSimone Consulting Engineers
160 Sansome Street, 16th Floor
San Francisco, CA 94104

DeSimone Project #4069

February 22, 2007

DESIMONE

NEW YORK
MIAMI
SAN FRANCISCO
NEW HAVEN
LAS VEGAS

September 28, 2006

City and County of San Francisco
Department of Building Inspection
1660 Mission Street, 2nd Floor
San Francisco, CA 94103

DeSimone Project #4069B
301 Mission Street

Attn: Mr. Hanson Tom, S.E.
Principal Engineer

Re: Summary of Meeting Between The City of San Francisco Department of Building Inspection and Millennium Partners and DeSimone Consulting Engineers, held on September 26, 2006, Re. 301 Mission Street (Permit No. 2002/1023/9696) - BauGrid® Reinforcement

Dear Hanson,

It was a pleasure meeting with you and your staff yesterday to discuss the repercussions on the 301 Mission Street project of a recent test performed by Professor Jack Moehle of UC Berkeley, in which a reinforced concrete column containing a sample of wire mesh reinforcement similar to BauGrid appears to have performed in an unexpected manner. As you have requested, we are pleased to offer the following summary of our discussions and the action items to which we mutually agreed.

SFDBI started the meeting by summarizing their concern regarding this issue, and their concern about the performance of the BauGrid product as a result of the recent test. DeSimone, as well as Millennium Partners, the project sponsor, indicated that they share the concern of SFDBI regarding this issue.

SFDBI suggested that additional testing might be the easiest way to resolve this issue. DeSimone expressed their concern that testing would not be a simple process since agreeing to an acceptable test and acceptance criteria would be the subject of much debate.

DeSimone also indicated that the recent test performed by Prof. Moehle differed from the conditions of the 301 Mission Street project in the several ways, including the following:

- The materials are not the same strength
- The reinforcing is not the BauGrid product that was manufactured by one of their certified facilities, nor was it of the same size or configuration as that product being used on our project.
- The loading conditions are different

DeSimone stated that additional testing to substantiate the integrity of the BauGrid product should not be required for the 301 Mission Street project for the following reasons:

- BauGrid is an ICC/ICBO approved product.
- BauGrid is being used on this project as a one-to-one substitution for cross ties in shear walls and columns in a manner consistent with the ICC/ICBO approvals
- BauGrid has been used on previous permitted and constructed projects in San Francisco in the same manner as our project without any requirements for additional testing
- The intended use of BauGrid on the 301 Mission Street project has been previously discussed and reviewed with both SFDBI and the Structural Peer Review Panel (SPRP). This discussion, which is included in the official SPRP binder, can be summarized as follows: The SPRP asked if additional testing of BauGrid was planned for the project, DeSimone indicated that it was not, and the SPRP indicated that our position was acceptable.

All parties discussed the letter dated September 19, 2006 from BauTech indicating that the materials tested by Prof. Moehle had not been subjected to their rigorous QA/QC procedures. SFDBI indicated that in light of these statements, they have reason to question the quality of the materials being delivered to the project site.

At the end of the meeting it was agreed that the following actions would be required to bring closure to this issue:

- DeSimone will submit revised construction drawings to SFDBI showing all structural elements where BauGrid is planned to be used on the project
- DeSimone will send a copy of the revised construction drawings to the individuals comprising the SPRP, Prof. Moehle and Hardip Pannu. At the request of SFDBI, an advance copy will be sent electronically to Prof. Moehle.
- The SPRP will be asked to review the drawings and to comment only and specifically on whether or not the drawings represent an appropriate implementation of the BauGrid product, i.e., is it being used as a one-to-one substitution for the cross ties previously shown on the permitted contract drawings.
- Millennium Partners and DeSimone will work with the project constructors to furnish SFDBI with the following information:
    - ♦ A copy of the BauTech QA/QC manual and procedures used for the production of BauGrid
    - ♦ A letter of certification from the testing and inspection agency responsible for overseeing the production of BauGrid for this project indicating that all QA/QC procedures are being followed
    - ♦ A letter from BauTech certifying that they have inspected the product being delivered to the project site and indicating that it has been manufactured in conformance with their own QA/QC procedures and with the ICBO approval documents.

We trust that you will agree that the above accurately summarizes the discussions and action items resulting from our recent meeting. If you have any comments on the above please do not hesitate to contact me directly. We look forward to working with you to resolve this issue to the satisfaction of SFDBI in the most expeditious way possible.

Very truly yours,

DESIMONE CONSULTING ENGINEERS, PLLC

Derrick D. Roorda, SE
Senior Associate

cc: Jonathan Rothstein, Steven Hood (Millennium Partners)
Jack Moehle, UC Berkeley
Hardip Pannu, Middlebrook + Louie

Comments on the Use of BauGrids as Shear Reinforcement

By: Murat Saatcioglu PhD., P.Eng.

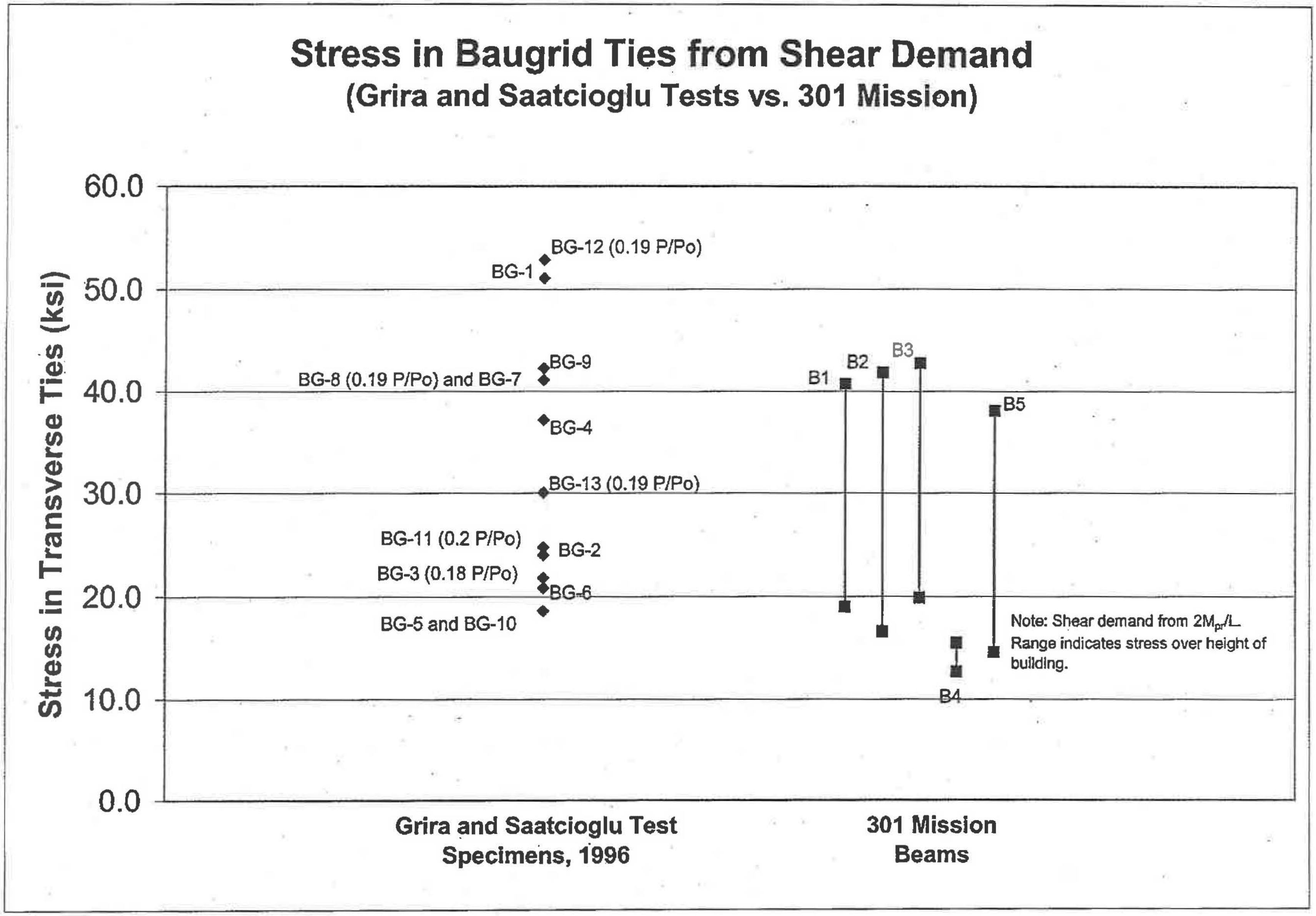
A total of 13 large scale column specimens were tested at the University of Ottawa, with BauGrids used as column transverse (confinement) reinforcement. The specimens had 350 mm (13.7 in) square cross-sections and 1645 mm (5.4 ft) shear span between the column footing and the point of inflection (of a first story building column). All the columns were flexure dominant elements. They were subjected to constant axial compression, either at approximately 20% P_o (20% of column concentric capacity) or 40% P_o, and tested under incrementally increasing inelastic deformation reversals (lateral shear force reversals). No beam tests were performed. In the absence of beam test results, column test data obtained under a relatively low axial load of 20% P_o may be used, while keeping in mind that the effect of axial compression is to reduce ductility. Hence these results should provide a somewhat conservative perspective of BauGrid behavior under shear force reversals. Of the 13 columns tested, 10 had 4,900 psi concrete and the remaining three (BG-11, BG-12 and BG-13) had 11,800 psi concrete. Hysteretic relationships for all columns subjected to 20%P_o are included in the following pages. Also shown are sample strain gauge data recorded.

<u>Observations:</u>

- The seismic beam shear design forces required by ACI 318-05 is the larger of; i) shear force under factored earthquake loads and ii) shear associated with the formation of plastic hinges at the ends of the beams, with the latter often governing. Hence, one has to protect the beams against premature brittle shear failure prior to the development of probable moment resistances, computed with 1.25 f_y, which accounts for possible strain hardening in the longitudinal beam reinforcement and possible increases in moments and shears upon the formation of plastic hinges. In the columns tested, plastic hinges have formed and the specimens developed 4% to 7% lateral drift ratios, depending on the amount of confinement reinforcement. All the column specimens developed their inelastic flexural capacities (probable moment resistances) without any sign of shear failure.
- Normal Strength Concrete columns (BG-3 and BG-8) had approximately the same amount of confinement reinforcement required by ACI 318 (one had 30% more the other had 17% less) and they both developed 6% drift without any sign of failure in the columns and in the grids. The welds maintained their integrity until after the columns failed due to either the longitudinal bar rupturing in tension or the compression buckling and subsequent concrete crushing.
- High-Strength Concrete columns (BG-11, BG-12 and BG-13) had approximately 70%, 30% or 50% of the confinement steel required by ACI 318. BG-11, with about 70% of the ACI confinement steel requirement developed 6% drift with little or no degradation in flexural resistance and failed during 7% drift cycles due to the rupturing of longitudinal tension reinforcement. Transverse strains recorded on BauGrids showed yielding of the second grid at 2% drift. The grid developed

strains of 0.02%, 0.3%, 0.7% and 1% at the third cycles of 1%, 2%, 3% and 4% drift levels, respectively.

- HSC Column BG-12 (with 34% of confinement reinforcement required by ACI-318) developed 4% drift before failure. The yielding of longitudinal reinforcement and of the second grid was recorded during the first cycle of 2% drift. The strain in the grid increased to 0.6% during the third cycle of 2% drift. The strain further increased to 0.98% during the third cycle of 3% drift. The grid ruptured at 4% drift level, followed by the rupturing of the second grid at 5% drift. The compression bars buckled during the second cycle at 5% drift and the test was discontinued. Although shear cracks were observed on the side faces (parallel to the direction of loading), they were well controlled hairline cracks.
- HSC Column BG-13 (with 53% of confinement reinforcement required by ACI 318-05) showed similar behavior as BG-12. Strain Gauges #4 and #5 placed on the outer perimeter of the second grid indicated yielding during the first load excursion at 1% drift. Strain readings of 0.2%, 0.3% and 0.4% were recorded on the same grid at 0.5%, 1% and 2% drift ratios, respectively. This column did experience a wide diagonal tension crack above the plastic hinge region, as depicted in the attached figure (Fig. 5-51), indicating possible yielding of the grids due to shear. However the grids were able to control the crack and the column failure was due to flexure.
- It should be noted that the above observations are only valid for the BauGrids provided for the test program conducted at the University of Ottawa in 1996.


Stress in Baugrid Ties from Shear Demand
(Grira and Saatcioglu Tests vs. 301 Mission)
Stress in Transverse Ties (ksi)
60.0
50.0
40.0
30.0
20.0
10.0
0.0
BG-12 (0.19 P/Po)
BG-1
BG-9
BG-8 (0.19 P/Po) and BG-7
BG-4
BG-13 (0.19 P/Po)
BG-11 (0.2 P/Po)
BG-2
BG-3 (0.18 P/Po)
BG-6
BG-5 and BG-10
B1
B2
B3
B4
B5
Note: Shear demand from $2M_{pr}/L$. Range indicates stress over height of building.
Grira and Saatcioglu Test Specimens, 1996
301 Mission Beams

4 April 2007

Mr. Hanson Tom
City and County of San Francisco
Building Inspection Department
160 Mission Street
San Francisco, CA 94103-2414

Subject: Peer Review Panel Recommendation to Accept Baugrid Reinforcement as Beam Transverse Reinforcement in the 301 Mission Project

Dear Hanson:

We have received the Structural Calculations package dated 22 February 2007, prepared and submitted by DeSimone Consulting Engineers, under the direction of the Derrick Roorda, the Engineer of Record on the 301 Mission Street Project. The package is subtitled *Shear Capacity of Moment Frame Beams Reinforced with BauGrid*, which is the main focus of the package. The package contains a detailed evaluation of the reasons why BauGrids can be accepted as transverse reinforcement in this specific project, including calculations, test data, and opinions from an outside consultant, Murat Saatcioglu, who is an expert in the use of BauGrids.

It is our understanding that the use of BauGrids in the moment frame beams is not being considered as a one-for-one, equivalent replacement of conventional transverse reinforcement. Instead, it is our understanding that the use of BauGrids in the moment frame beams is being proposed on the basis of a performance approach. According to this approach, the use of Baugrids is deemed acceptable if the calculated performance of the buildings is equivalent to or better than the performance anticipated if those buildings were reinforced with conventional transverse reinforcement.

With this understanding, and after review of the information provided in the 22 February 2007 package as well as previous information provided to us about the design of these buildings, it is our opinion that the use of BauGrids in the moment frame beams is acceptable as proposed.

Although we have exercised usual and customary professional care in providing this review, we have not independently verified the accuracy of the calculations provided by DeSimone. Our professional opinions are based on their calculations and further the responsibility of the structural design remains fully with the Engineer of Record.

Respectfully,

Jack Moehle

Jack P. Moehle

Hardip Pantu

Hardip Pantu

Stress-Strain Relationship of 10 ksi Concrete Confined by Baugrids

Confinement efficiency parameter:

$$\mathbf{k_2 = 0.15\sqrt{\frac{b_c}{s}\frac{b_c}{s_\ell}} = 0.15\sqrt{\frac{12}{4}\frac{12}{4}} = 0.45}$$

b_c : center to center core dimension = 12 in

s_ℓ: spacing of crossties = 4 in.

Average lateral pressure (at yielding of transverse reinforcement):

$\mathbf{f_\ell}$= 4 x (0.2sq in) (83ksi)/ (12 in x 4 in) = 1.38 ksi (9.54 MPa)

Equivalent uniform pressure:

$\mathbf{f_{\ell e}} = k_2\ \mathbf{f_\ell}$ = (0.45) (1.38) = 0.62 ksi (4.29 MPa)

Confined Concrete Strength

$$\mathbf{f'_{cc} = f'_{co} + k_1 f_{\ell e}}$$

$k_l = 6.7\ (f_{le})^{-0.17}$
$k_l = 6.7\ (4.29)^{-0.17} = 5.23$ (note that this equation is unit dependent and must be used with lateral pressure in MPa)

$f'_{co} = f'_c$ x 0.9 = 10 x 0.9 = 9 ksi (in-place strength of concrete in member – as opposed to cylinder strength)

f'_{cc} = 9 ksi + 5.23 (0.62) = 12.2 ksi (confined concrete strength in the core)

Ratio of additional strength due to confinement to in-place strength of unconfined concrete (K);

$K = k_l\ \mathbf{f_{\ell e}} / f'_{co}$ = 5.23 (0.62) / 9.0 = 0.36 (36% more strength due to confinement)

HSC adjustment factors; k_3 and k_4 (strengths are both in MPa):

$k_3 = 40/f'_{co}$ = 40/(62) = 0.64 (strengths in MPa)
$k_4 = f_{yt}/500$ = 572/500 = 1.14 (strengths in MPa)

Unconfined concrete strains at peak stress and at 85% of peak beyond the peak stress:

$\varepsilon_{01} = 0.0028 - 0.0008\ k_3 = 0.00229$

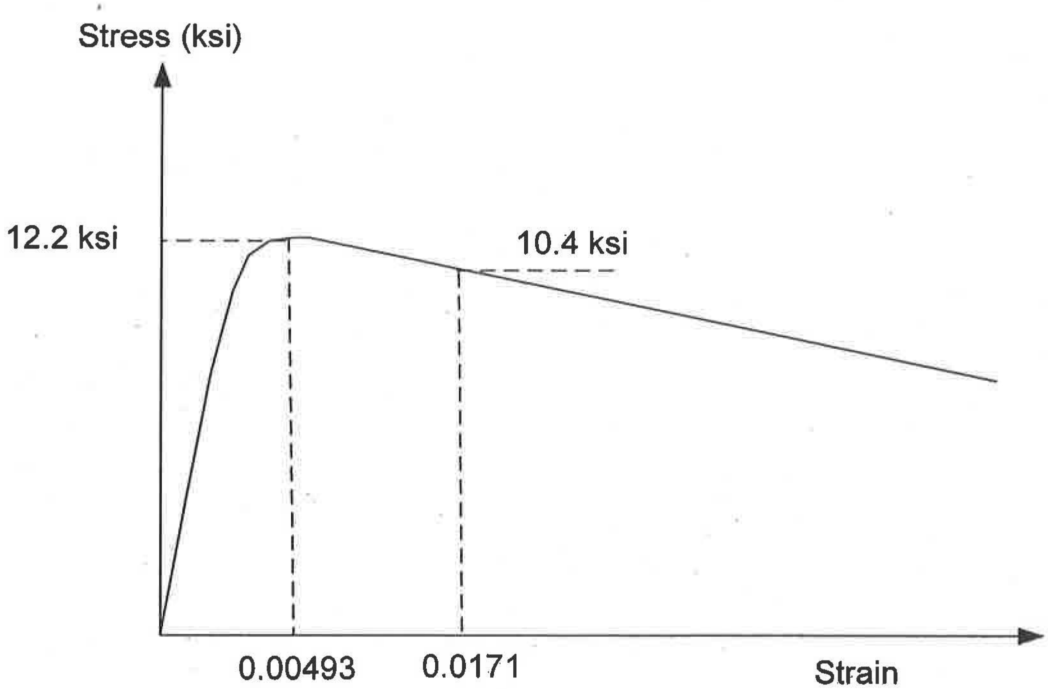
$\varepsilon_{85} = \varepsilon_{01} + 0.0018\,(k_3)^2 = 0.0030$

Confined concrete strains:

$\varepsilon_1 = \varepsilon_{01}\,(1+5\,k_3\,K) = 0.00229\,[1 + 5(0.64)(0.36)] = 0.00493$

$\varepsilon_{85} = 260\,k_3\,\rho_c\,\varepsilon_1\,[1+0.5k_2(k_4-1)] + \varepsilon_{085}$

$= (260)(0.64)(4*0.2/(12*4))\,(0.00493)[1+0.5(0.45)(1.14-1.0)] + 0.0030 = 0.0171$



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Project: 301 MISSION

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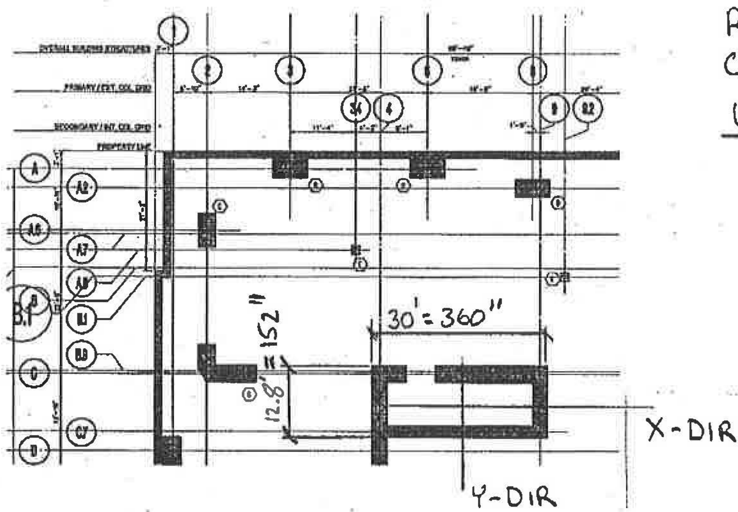
Project No.: 4069

Date: 11/21/06

Item: STRAIN DEMAND PER UBC 1921.6.6.5

By: NJR

Ch'kd:



NORTH CORE FOR 301

ROOF DISPLACEMENT IN CONFORMANCE WITH UBC 1921.6.6.5

$$\Delta_{m_{y\text{-DIR}}} = 73'' = \Delta_{t_y}$$

$$\Delta_{m_{x\text{-DIR}}} = 78'' = \Delta_{t_x}$$

ASSUMPTIONS:

$$\phi_y = \frac{0.003}{l_w} = \begin{cases} \phi_{y_{y\text{-AXIS}}} = 8.3 \times 10^{-6}\ 1/in \\ \phi_{y_{x\text{-AXIS}}} = 19.7 \times 10^{-6}\ 1/in \end{cases}$$

$$l_p = \frac{l_w}{2} = \begin{cases} l_{p_x} = \frac{l_{w_x}}{2} = \frac{360''}{2} = 180'' \\ l_{p_y} = \frac{l_{w_y}}{2} = \frac{152''}{2} = 76'' \end{cases}$$

GIVENS

$h_w = 605\ ft = 7260\ in$

$l_{w_y} = 152''$

$l_{w_x} = 360''$

$P'_{u_y} = 1.2(28{,}800^k) + 0.5(5600^k) + 13{,}000^k$

$= 37{,}400 + 13{,}000 = 50{,}400^k$

$P'_{u_x} = 37{,}400 + 600^k$

$= 38{,}000^k$

Δ YIELD IN X-DIR EQ (V_{BASE} = 6268 kips)

BASED ON GROSS SECTION ELASTIC MODEL:

$\Delta_E = 9.7''$

$\left.\begin{array}{l} V_E = 2900 \text{ kips} \\ M_E = 7400 \text{ kip-ft} \end{array}\right\}$ MAX FORCES IN OUTRIGGER LINKS

YIELD IN OUTRIGGERS WILL START WHEN NOMINAL CAPACITIES ARE REACHED

$V_n' = 4988 \text{ kips} \quad \text{SCALE} = \frac{V_n'}{V_E} = \frac{4988}{2900} = 1.72$ ← GOVERNS

$M_n' = 15808 \text{ kip-ft} \quad \text{SCALE} = \frac{M_n'}{M_E} = \frac{15808}{7400} = 2.13$

$\therefore$ SCALE FOR FIRST YIELD IS 1.7

$\Delta_y = \text{SCALE} \cdot \Delta_E$

$= 1.7 \cdot 9.7''$

$= 16.5''$

$\boxed{\Delta_{y_x} = 16.5''}$

Δ YIELD IN Y-DIR EQ (V_{BASE} = 6268 kips)

BASED ON GROSS SECTION ELASTIC MODEL:

$\Delta_E = 8.7''$

$V_E = 330^{kips}$ (SHEAR IN STEEL LINK BEAM)

T_E = 12,900 kips (UPLIFT IN TENSION FLANGE OF CORE)

M_E = 41,000 kip-ft (MOMENT IN COMP. FLANGE OF CORE.)

→ WHAT ARE THE SCALE FACTORS TO CAUSE THE ABOVE ELEMENTS TO YIELD?

SHEAR LINK: $V_n' = 620$ kips ∴ $SCALE = \frac{V_n'}{V_E} = \frac{620^k}{330^k} = 1.88$ ← GOVERNS

TENSION FLANGE: $T_n' = \overset{(GRAV)}{28{,}000} + 6000k\,(\text{UPLIFT}) = 35{,}000$ kips
@ M_E = 41,000

∴ $SCALE = \frac{35{,}000 \text{ kips}}{12{,}900 \text{ kips}} = 2.71$

MOMENT IN COMP. FLANGE: M_n' = 400,000 k-ft

$SCALE = \frac{M_n'}{M_E} = \frac{400{,}000}{41{,}000} = 9.7$

∴ SCALE @ FIRST YIELD IS 1.9

$\Delta_y = 1.9\,\Delta_E = 1.9(8.7'')$
$= 16.5''$

$\boxed{\Delta_y = 16.5''}$

DETERMINE INELASTIC DEFLECTION

X DIR EQ: $\Delta_{i_x} = \Delta_{t_x} - \Delta_{y_x}$

$= 78'' - 16.5''$

$= 61.5''$

$\boxed{\Delta_{i_x} = 61.5''}$

Y DIR EQ: $\Delta_{i_y} = \Delta_{t_y} - \Delta_{y_y}$

$= 73'' - 16.5''$

$= 56.5''$

$\boxed{\Delta_{i_y} = 56.5''}$

DESIMONE

Project		Page 5	Of 9
Project No.		Date 11/21/06	
Item		By NJR	Ch'kd

TOTAL CURVATURE DEMAND

$$\phi_{t_{y\text{-AXIS}}} = \frac{\Delta_{ix}}{\left(h_w - \frac{l_p}{2}\right) l_p} + \phi_{y_{y\text{-AXIS}}}$$

$$= \frac{61.5''}{(7260'' - 180''/2)(180'')} + 8.3 \times 10^{-6}\ 1/in$$

$$= 47.6 \times 10^{-6}\ 1/in + 8.3 \times 10^{-6}\ 1/in$$

$$= 55.9 \times 10^{-6}\ 1/in$$

$$\boxed{\phi_{t_{y\text{-AXIS}}} = 55.9 \times 10^{-6}\ 1/in}$$

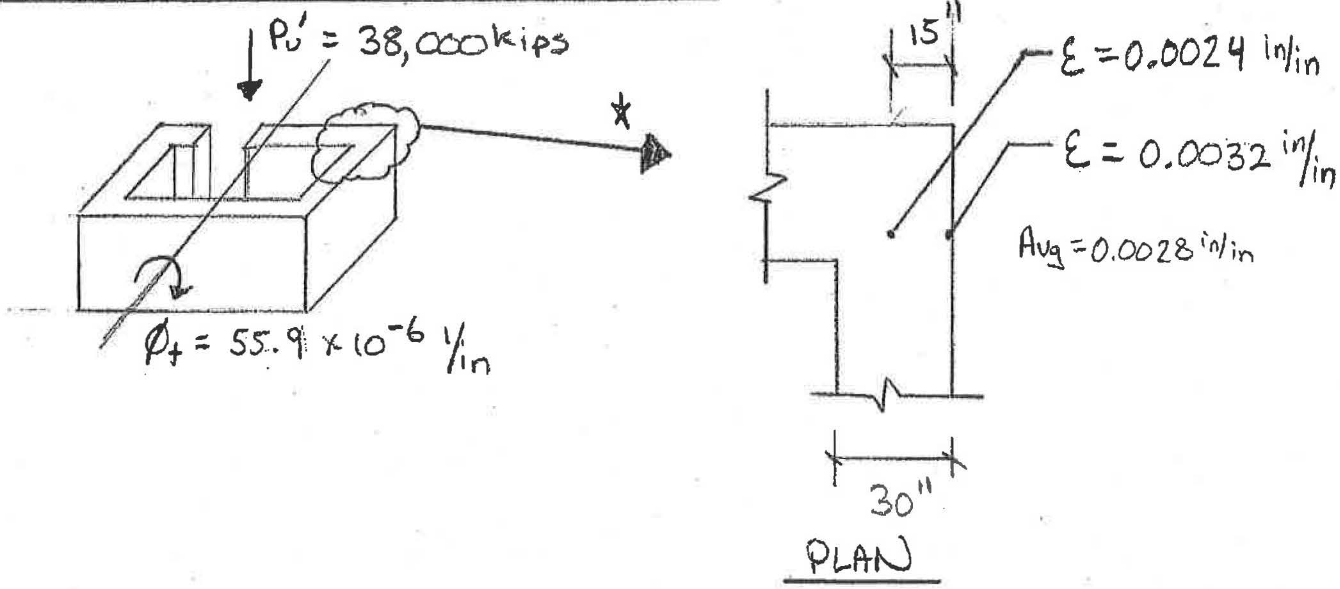
$$\phi_{t_{X\text{-AXIS}}} = \frac{56.5''}{\left(7260'' - \frac{76''}{2}\right)(76'')} + 19.7 \times 10^{-6}\ 1/in$$

$$= 102.9 \times 10^{-6}\ 1/in + 19.7 \times 10^{-6}\ 1/in$$

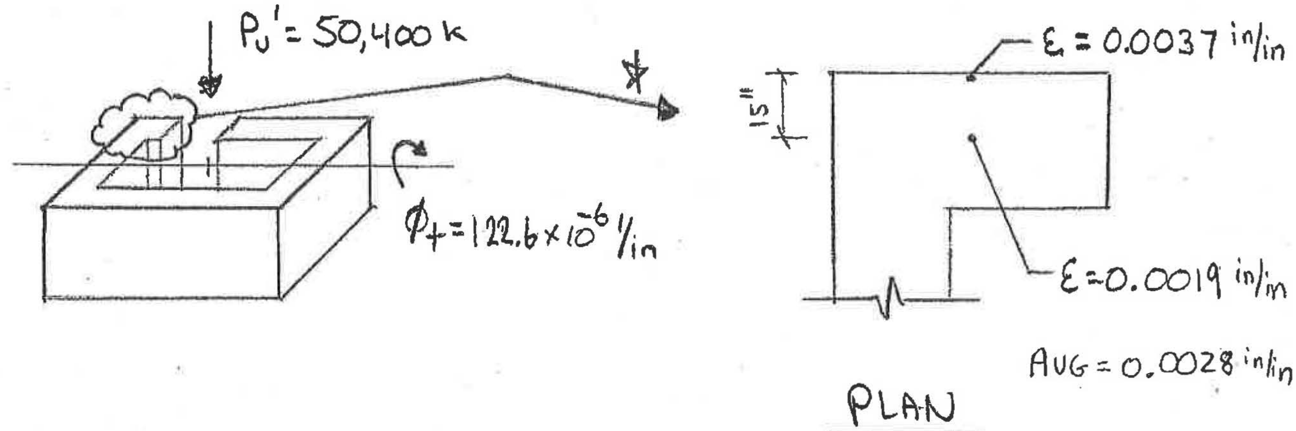
$$= 122.6 \times 10^{-6}\ 1/in$$

$$\boxed{\phi_{t_{X\text{-AXIS}}} = 122.6 \times 10^{-6}\ 1/in}$$

DETERMINATION OF STRAIN DEMAND



$$\boxed{\varepsilon_{AVG_x} = 0.0028 \text{ in/in}}$$



$$\boxed{\varepsilon_{AVG_y} = 0.0028 \text{ in/in}}$$

* STRAINS DETERMINED BY STRAIN COMPATIBILITY ANALYIS USING THE SOFTWARE XTRACT. P_U' WAS ASSUMED CONSTANT. SEE PAGE 8 AND 9.

ORTHOGONAL EFFECTS.

SIMILAR TO WHAT IS SUGGESTED IN UBC 1633.1, ORTHOGONAL EFFECTS WILL BE ACCOUNTED FOR BY USING 100% + 30% COMBINATION RULE.

$$\varepsilon_{TOTAL} = 1.00(\varepsilon_{AVG_x}) + 0.30(\varepsilon_{AVG_y})$$

$$= 1.0(0.0028\ in/in) + 0.30(0.0028\ in/in)$$

$$= 0.0036\ in/in$$

MCE EFFECTS

$$\text{ASSUME} \quad \frac{DEMAND_{MCE}}{DEMAND_{DBE}} \simeq 1.6$$

$$\therefore TOTAL_{MCE}\ \varepsilon_{DEMAND} \simeq 1.6(0.0036\ in/in)$$

$$= 0.0058\ in/in$$

THE MAXIMUM DEMAND THAT CAN BE EXPECTED FOR THE MCE EVENT INCLUDING ORTHOGONAL EFFECT IS TAKEN TO BE

$$\boxed{\varepsilon_{MAX\ DEMAND} = 0.006\ in/in}$$

DESIMONE
11/21/06 NJR

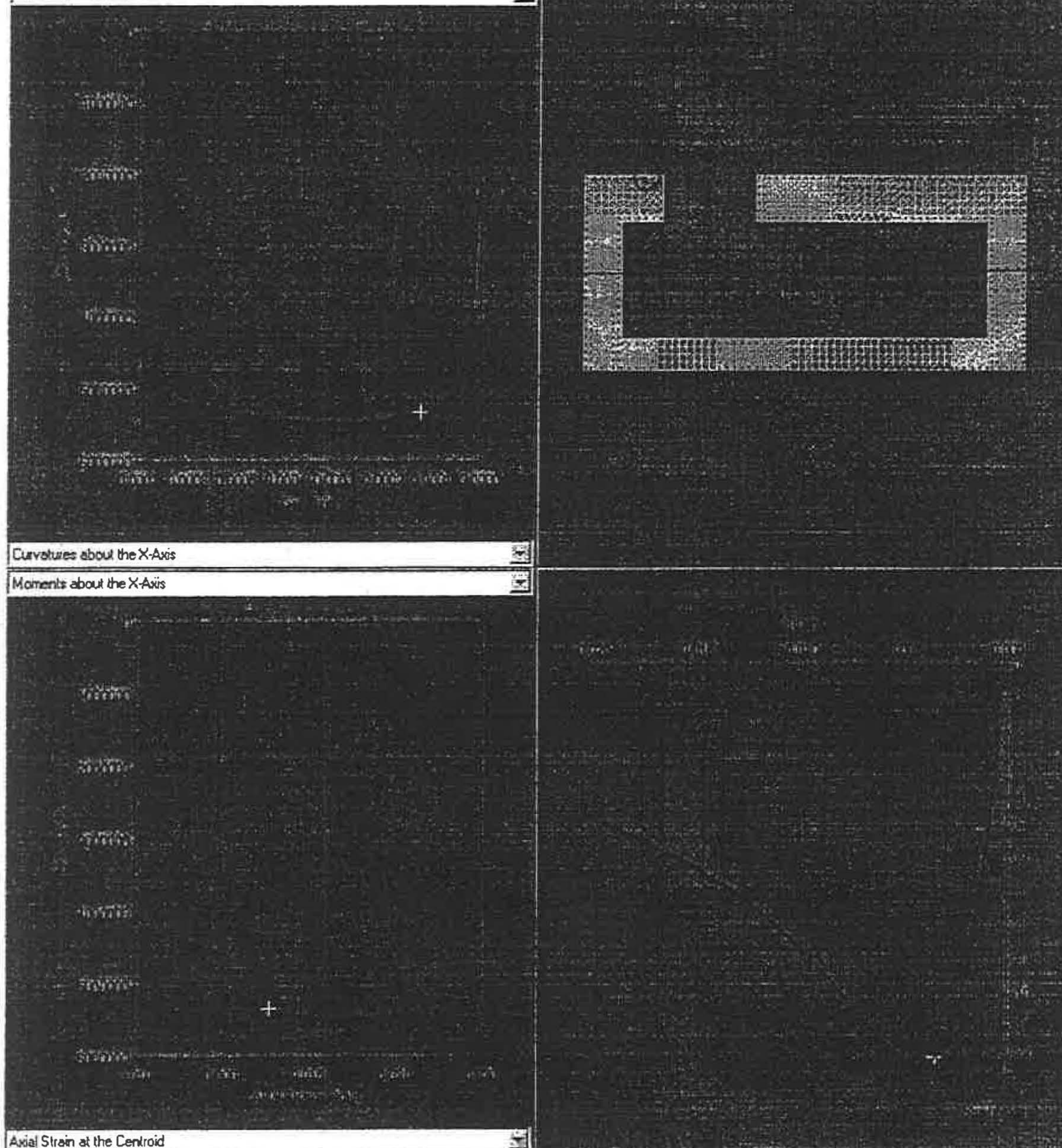
Section1 - Pu_50400

26 — LOAD STEP

kip-in

PG 8 OF 9

Moments about the X-Axis



Curvatures about the X-Axis

Moments about the X-Axis

Axial Strain at the Centroid

Select Data to View

- Fiber/Bar Property
- Section Analysis Property
- Material Totals

P_u'

Section Property	Value
X Centroid	8.719 in
Y Centroid	76.88 in
Axial Force	-50.40E+3 kips
Moment xx	-5.313E+6 kip-in
Moment yy	625.4E+3 kip-in
Principal Moment	5.349E+6 kip-in
Axial Strain	5.605E-3 Ten
Curvature xx	-.1214E-3 1/in
Curvature yy	.6432E-17 1/in
Principal Curvature	.1214E-3 1/in
Eccentricity x	12.41 in
Eccentricity y	105.4 in
Centroid to NA	46.16 in

CURVATURE
$\simeq 122 \times 10^{-6}$

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DeSimone
11/21/06 NJR

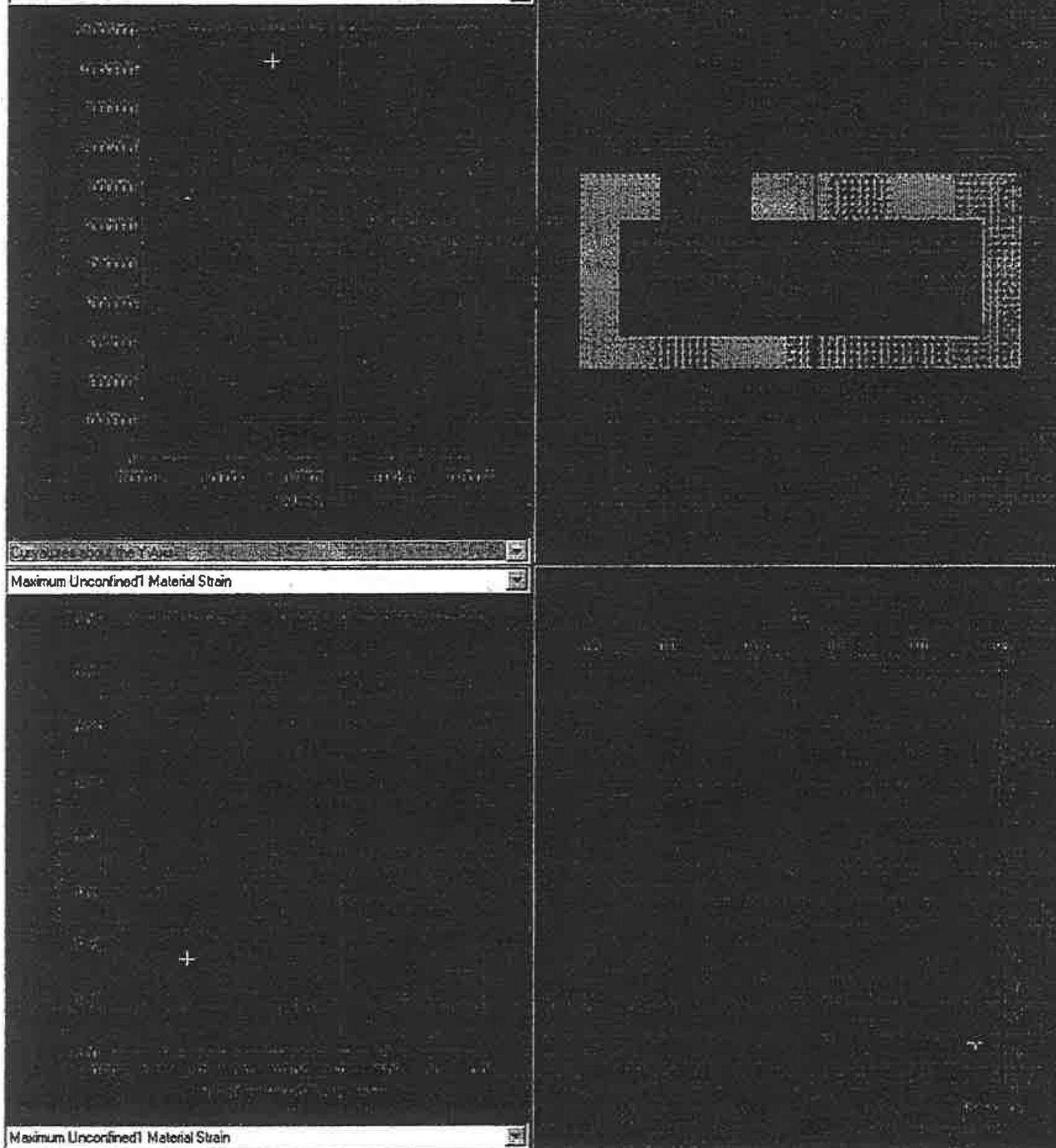
Section1 - Pu_38000

27 — LOADSTEP

kip-in

PG 9 OF 9

Moments about the Y-Axis



Curvatures about the Y-Axis

Maximum Unconfined1 Material Strain

Maximum Unconfined1 Material Strain

Select Data to View

- Fiber/Bar Property
- Section Analysis Property
- Material Totals

P_u

Section Property	Value
X Centroid	8.719 in
Y Centroid	76.88 in
Axial Force	-38.00E+3 kips
Moment xx	-109.8E+3 kip-in
Moment yy	1.02E+7 kip-in
Principal Moment	1.02E+7 kip-in
Axial Strain	6.686E-3 Ten
Curvature xx	.1537E-17 1/in
Curvature yy	58.04E-6 1/in
Principal Curvature	58.04E-6 1/in
Eccentricity x	269.0 in
Eccentricity y	2.889 in
Centroid to NA	115.2 in

CURVATURE
$\simeq 55.9 \times 10^{-6}$ 1/in

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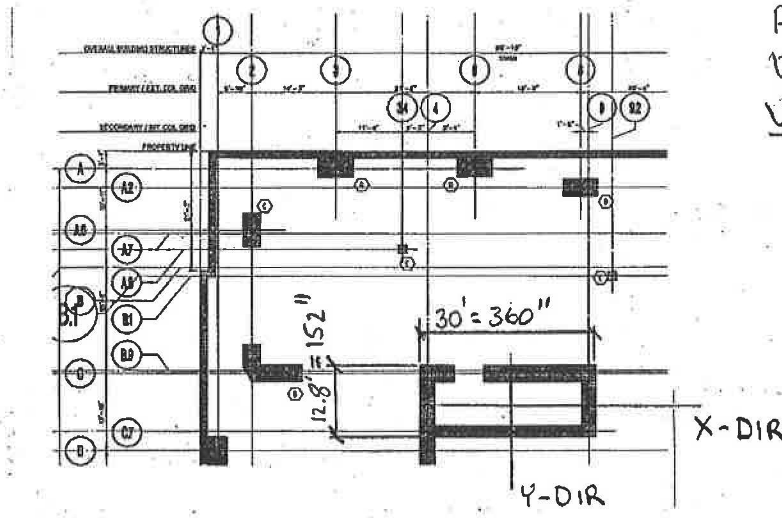


NORTH CORE FOR 301

ROOF DISPLACEMENTS OBTAINED USING ELASTIC R.S.A. WITH UNREDUCED MCE SPECTRA

$$\Delta_{MCE_{Y\text{-}DIR}} = 85'' = \Delta_{t_y}$$

$$\Delta_{MCE_{X\text{-}DIR}} = 97'' = \Delta_{t_x}$$

GIVENS

BUILDING OVERSTRENGTH PER PEER REVIEW J.M. COMMENT 7

$\phi = 1.75$

$P_{E_y} = 13{,}000$ kips

$h_w = 605\text{ ft} = 7260\text{ in}$

$l_{w_y} = 152''$ $\quad l_{w_x} = 360\text{ in}$

$P_{ULT_y} = \phi \cdot P_{E_y} = (1.75)(13{,}000)$
$= 22{,}750$ kips

$P'_{u_y} = 1.2(28{,}800^k) + 0.5(5600^k) + 22{,}750^k$
$= 37{,}400 + 22{,}750 = 60{,}150^k$

$P'_{u_x} = 37{,}400 + 600^k$
$= 38{,}000^k$

ASSUMPTIONS:

$$\phi_y = \frac{0.003}{l_w} = \begin{cases} \phi_{y_{y\text{-AXIS}}} = 8.3 \times 10^{-6}\ 1/\text{in} \\ \phi_{y_{X\text{-AXIS}}} = 19.7 \times 10^{-6}\ 1/\text{in} \end{cases}$$

$$l_p = \frac{l_w}{2} = \begin{cases} l_{p_x} = \frac{l_{w_x}}{2} = \frac{360''}{2} = 180'' \\ l_{p_y} = \frac{l_{w_y}}{2} = \frac{152''}{2} = 76'' \end{cases}$$

Δ YIELD IN X-DIR EQ ($V_{BASE} = 6268$ kips)

BASED ON GROSS SECTION ELASTIC MODEL:

$\Delta_E = 9.7''$

$\left.\begin{array}{l} V_E = 2900 \text{ kips} \\ M_E = 7400 \text{ kip-ft} \end{array}\right\}$ MAX FORCES IN OUTRIGGER LINKS

YIELD IN OUTRIGGERS WILL START WHEN NOMINAL CAPACITIES ARE REACHED

$V_n' = 4988 \text{ kips} \quad \text{SCALE} = \frac{V_n'}{V_E} = \frac{4988}{2900} = 1.72$ ← GOVERNS

$M_n' = 15808 \text{ kip-ft} \quad \text{SCALE} = \frac{M_n'}{M_E} = \frac{15808}{7400} = 2.13$

∴ SCALE FOR FIRST YIELD IS 1.7

$\Delta_y = \text{SCALE} \cdot \Delta_E$

$= 1.7 \cdot 9.7''$

$= 16.5''$

$\boxed{\Delta_{y_x} = 16.5''}$

DESIMONE

Δ YIELD IN Y-DIR EQ ($V_{BASE} = 6268$ kips)

BASED ON GROSS SECTION ELASTIC MODEL:

$\Delta_E = 8.7''$

$V_E = 330^{kips}$ (SHEAR IN STEEL LINK BEAM)

$T_E \approx 12{,}900$ kips (UPLIFT IN TENSION FLANGE OF CORE)

$M_E = 41{,}000$ kip-ft (MOMENT IN COMP. FLANGE OF CORE.)

→ WHAT ARE THE SCALE FACTORS TO CAUSE THE ABOVE ELEMENTS TO YIELD?

SHEAR LINK: $V_n' = 620^{kips}$ $\therefore$ SCALE $= \frac{V_n'}{V_E} = \frac{620k}{330k} = 1.88$ ← GOVERNS

TENSION FLANGE: $T_n' = \overset{(GRAV)}{28{,}000} + 6000k$ (UPLIFT) $= 35{,}000$ kips
@ $M_E = 41{,}000$

$\therefore$ SCALE $= \frac{35{,}000 \text{ kips}}{12{,}900 \text{ kips}} = 2.71$

MOMENT IN COMP. FLANGE: $M_n' = 400{,}000$ k-ft

SCALE $= \frac{M_n'}{M_E} = \frac{400{,}000}{41{,}000} = 9.7$

$\therefore$ SCALE @ FIRST YIELD IS 1.9

$\Delta_y = 1.9\,\Delta_E = 1.9(8.7'')$
$= 16.5''$

$\boxed{\Delta_{y_y} = 16.5''}$

DESIMONE

PROJECT

PROJECT NO.

ITEM

DATE 12/1/06

BY CHK'D

DETERMINE INELASTIC DEFLECTION

X DIR EQ: $\Delta_{i_x} = \Delta_{t_x} - \Delta_{y_x}$

$= 97'' - 16.5''$

$= 80.5''$

$\boxed{\Delta_{i_x} = 80.5''}$

Y DIR EQ: $\Delta_{i_y} = \Delta_{t_y} - \Delta_{y_y}$

$= 85'' - 16.5''$

$= 68.5''$

$\boxed{\Delta_{i_y} = 68.5''}$

TOTAL CURVATURE DEMAND

$$\phi_{t_{y\text{-AXIS}}} = \frac{\Delta_{ix}}{\left(h_w - \frac{l_p}{2}\right) l_p} + \phi y_{y\text{-AXIS}}$$

$$= \frac{80.5''}{(7260'' - 180''/2)(180'')} + 8.3\times10^{-6}\ 1/in$$

$$= 62.3\times10^{-6}\ 1/in + 8.3\times10^{-6}\ 1/in$$

$$= 70.7\times10^{-6}\ 1/in$$

$$\boxed{\phi_{t_{y\text{-AXIS}}} = 70.7\times10^{-6}\ 1/in}$$

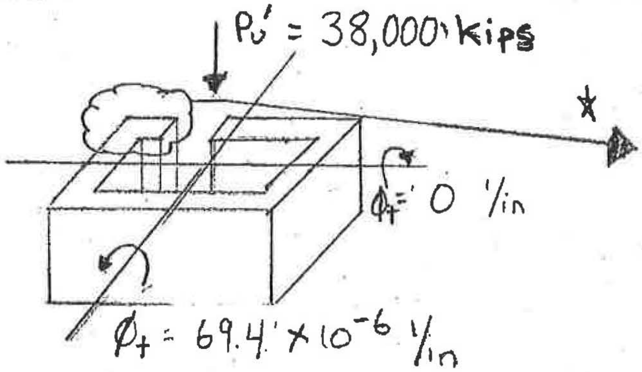
$$\phi_{t_{X\text{-AXIS}}} = \frac{68.5''}{\left(7260'' - \frac{76''}{2}\right)(76'')} + 19.7\times10^{-6}\ 1/in$$

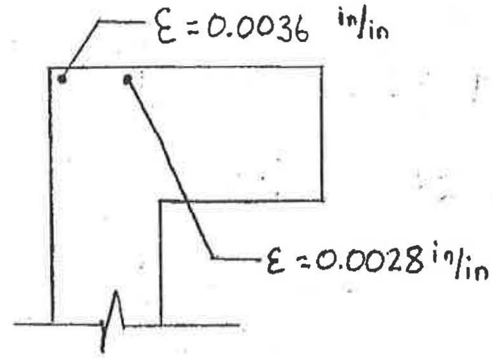
$$= 124.8\times10^{-6}\ 1/in + 19.7\times10^{-6}\ 1/in$$

$$= 144.5\times10^{-6}\ 1/in$$

$$\boxed{\phi_{t_{X\text{-AXIS}}} = 144.5\times10^{-6}\ 1/in}$$

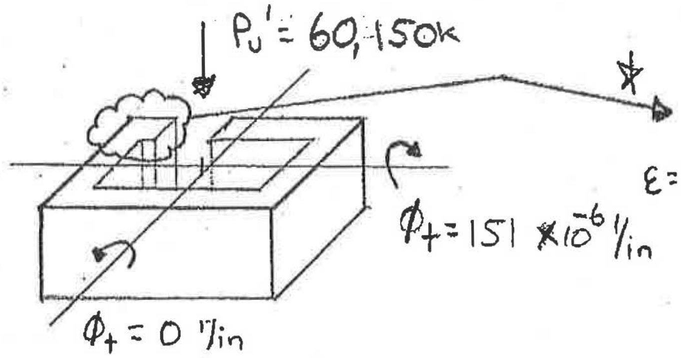
DETERMINATION OF STRAIN DEMAND

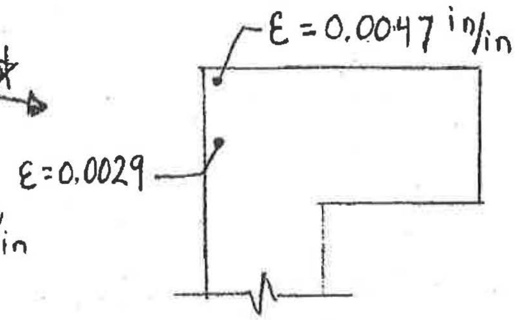




PLAN

$$\boxed{\varepsilon_{max\,x} = 0.0036 \text{ in/in}}$$





PLAN

$$\boxed{\varepsilon_{max\,y} = 0.0047 \text{ in/in}}$$

★ STRAINS DETERMINED BY STRAIN COMPATIBILITY ANALYIS USING THE SOFTWARE XTRACT. CONFINED CONCRETE MATERIAL MODEL BY SAATCIOGLU.

DESIMONE

ORTHOGONAL EFFECTS

1) 30% (X-LOAD) + 100% (Y-LOAD)

$= 30\%(\varepsilon_{MAX_X}) + 100\%(\varepsilon_{MAX_Y})$

$= 0.3(0.0036) + 1.0(0.0047)$

$= 0.0058$ in/in ← GOVERNS

2) 100% (X-LOAD) + 30% (Y-LOAD)

$= 100\%(\varepsilon_{MAX_X}) + 30\%(\varepsilon_{MAX_Y})$

$= 1.0(0.0036) + 0.3(0.0047)$

$= 0.0050$ in/in

$\boxed{\therefore \text{ TEST BAUGRID TO } \varepsilon = 0.006 \text{ in/in}}$

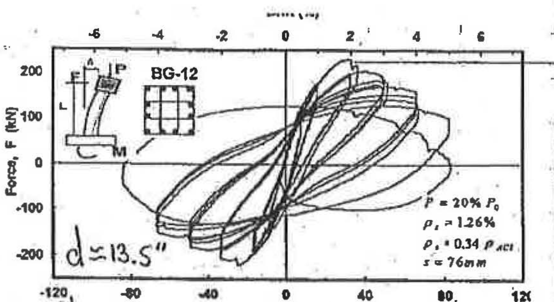
DESIMONE

PROJECT 301 MISSION — PAGE 1 OF 4

PROJECT NO. 4069B — DATE 12/22/06

ITEM SUMMARY OF CALCULATIONS PROVIDED FOR BEAM BAUGRID — BY NJR — CHK'D

STRESS FOR SAATCIOGLU TEST



$f'_c = 11{,}800$ psi

a) Hysteretic force-displacement relationship

MAX SHEAR OBTAINED ≅ 225 kN (50.6 kips)

$$V_s = \frac{A_s}{S}\, d\, f_s$$

$$\Rightarrow f_s = \frac{V_s \cdot S}{A_s \cdot d}$$

BG-12

$S = 3$ in

$A_s = (0.05\,in^2)(4) = 0.21\,in^2$

$$\therefore f_s = \frac{(50.6k)(3\,in)}{(0.21\,in^2)(13.5\,in)} = 53\text{ ksi}$$

NOTES: 1 - SINCE WELD SHEAR IS THE ACCEPTANCE CRITERIA, THE SHEAR DEMAND WAS NOT REDUCED BY V_c.

~~2 - BG-1 THROUG BG-13 WERE DONE SIMILARILY.~~

~~VALUE REPORTED ON PLOT:~~

$\boxed{\text{BG-12} \quad F_s = 53\text{ ksi}}$

STRESS FOR 301 MISSION

BEAM B3 AT LEVEL 1

$$V_s = \frac{2M_{pr}}{L} - V_c \;; \quad V_c = 1 \cdot \sqrt{f'_c}\, b_w d$$

$$= 619^k - 84^k$$

$$= 535^k$$

(PAPERS BY PRIESTLEY AND OTHERS SHOW $1\sqrt{f'_c}$ to be the lower bound for CONCRETE SHEAR CONTRIBUTION TO STRENGTH.)

NOTE: Mpr per ACI 21.0

STRESS FOR 301 MISSION (CONT.)

$$f_s = \frac{V_s \cdot S}{A_s \cdot d} \quad ; \quad S = 4.5'' \quad d = 35''$$

$$A_s = 6\,(0.31\,in^2) = 1.86\,in^2$$

$$= \frac{(535^k)(4.5'')}{(1.86\,in^2)(35'')}$$

$$= 37.0\ ksi$$

VALUE REPORTED ON PLOT

$\boxed{\text{B3 UPPER BOUND} \Rightarrow f_s = 37\ ksi}$

STRESS FOR WELD SHEAR

$$\text{STRESS} = \frac{\text{ULTIMATE LOAD}}{\text{AREA OF \# 5}}$$

Ultimate loads (N)	Ultimate loads (lb)	Area of #5 (in^2)	Stress (ksi)	
68228	15338	0.31	49	
62428	14034	0.31	45	
70257	15794	0.31	51	← UPPER BOUND
67171	15101	0.31	49	
60114	13514	0.31	44	
61628	13855	0.31	45	
61400	13803	0.31	45	
60142	13520	0.31	44	← LOWER BOUND

LOADS FROM ASTM 185 WELD SHEAR TEST. THE WELD FAILED AT THESE LOADS.

DESIMONE

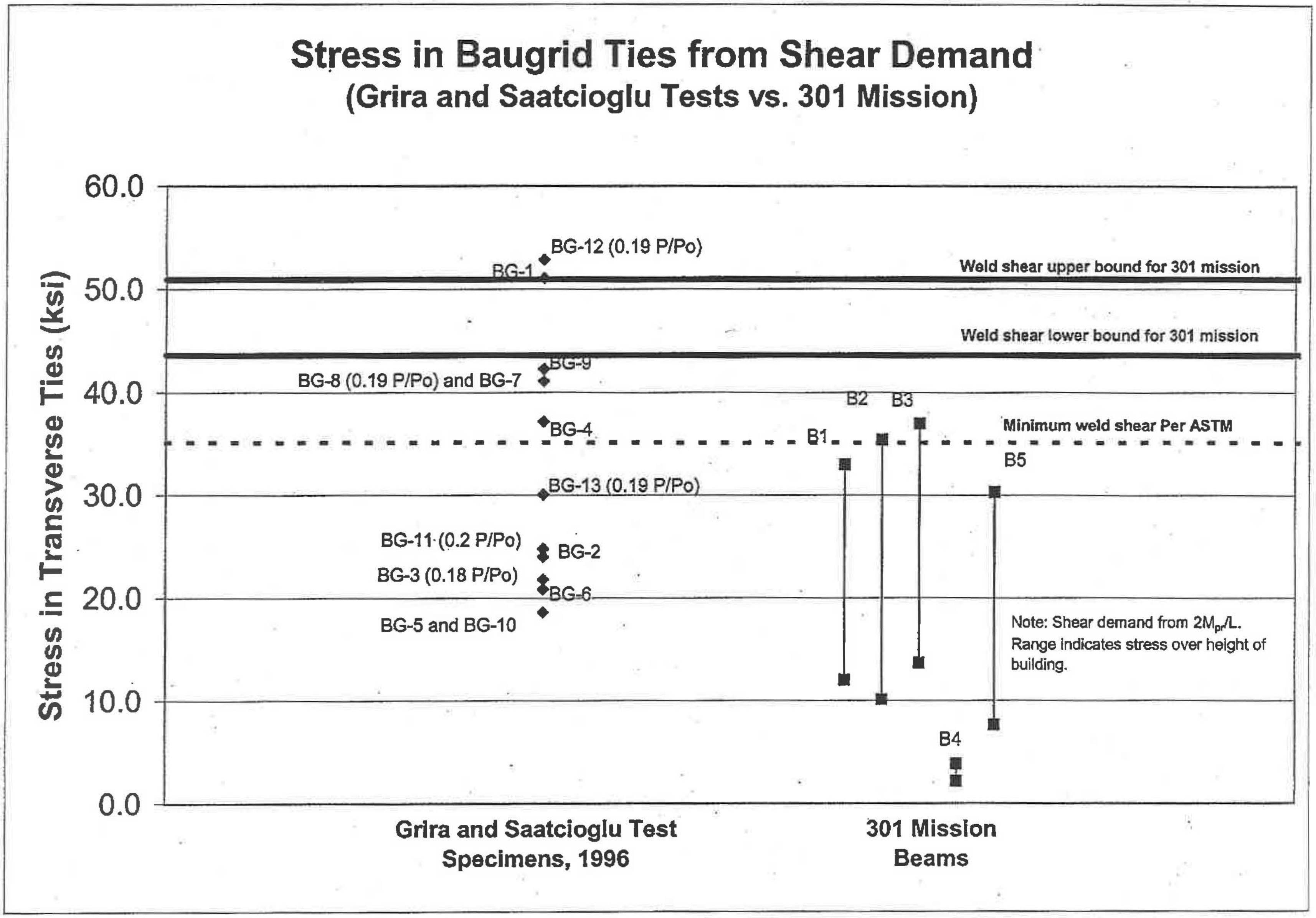
PROJECT NO.
DATE 12/22/06
ITEM
BY NJR CHK'D

CRITERIA AND CONCLUSION

CRITERIA: THE STRESS DEMANDS IN THE 301 MISSION BEAM BAUGRID SHOULD BE LESS THAN THE STRESS REQUIRED TO BREAK THE WELD PER THE ASTM 185 PROCEDURE.

CONCLUSIONS: SINCE THE DEMAND IN THE 301 MISSION BEAM BAUGRID IS LESS THAN THE FORCE REQUIRED TO BREAK THE WELDS, THE BEAM BAUGRID IS OK FOR USE ON THE 301 MISSION PROJECT.

THE SAATCIOGLU BAUGRID TESTS EXHIBIT DUCTILE BEHAVIOR AT BAUGRID STRESS LEVELS SIMILAR TO STRESS LEVELS EXPECTED ON THE 301 MISSION PROJECT.



P4606-20061221-NJR-Murat test vs 301 mission shear demand on ties-with conc and weld shear.xls

Background

Welded reinforcement grids are available in various sizes and shapes that are suitable for use in structural members as concrete reinforcement. Research conducted on reinforced concrete columns, shear walls, and beams indicate that welded grids offer superior performance and easy cage assembly when used as transverse reinforcement. The grid pattern improves concrete confinement and results in enhanced deformability in the inelastic range of deformations. This feature makes welded grids especially suitable for seismic resistant structures.

The specific grid product being used for the 301 Mission Street project is BauGrid, manufactured by BauTech, which has been approved for use by the ICBO Evaluation Service, Inc. as documented on ER-5192 dated August 1, 2000. BauTech maintains it's approved ICBO status by adhering to strict quality control requirements, which are audited quarterly by an independent inspection and testing agency, Smith Emery Laboratories. BauTech's Quality Assurance Program requires daily production sampling and testing to assure the quality of the product, and those tests have been duplicated on the specific batch of material utilized in these test columns.

Mission Street Development LLC. has been asked by the City of San Francisco Department of Building Inspection (SFDBI) to perform some additional tests to confirm the performance of BauGrid specific to the 301 Mission Street project.

DeSimone proposes to demonstrate that BauGrid is acceptable for use on the 301 Mission Street project through a testing program to be executed at UC Berkeley under the direction of Prof. Jack Moehle. The testing program outlined herein has been developed in response to the December 6, 2006 letter from SFDBI, and has been agreed to by the project SPRP and by SFDBI.

Test Specimens

- A total of six specimens shall be built and instrumented in accordance with SK-01.
- Preliminary Test. Specimens A1, A2, and A3.
- City Test. Specimens B1, B2, and B3.

Concrete Placement and Cylinders

- Concrete with expected 28-day strength of 10,000 shall be placed in all six specimens on the same day.
  A total of forty (40) concrete cylinders shall be taken from the same batch of concrete for the purpose of determining compressive strength.
- Two (2) cylinders shall be tested on the 5th day after concrete placement and on each day thereafter until the concrete strength reaches 10,000 psi.
- Two (2) additional cylinders shall be tested at 28, 56, and 90 days after concrete placement.

Test Procedure

- Each specimen shall be subjected to monotonic concentric axial compression loading.
- Data shall be continuously gathered and recorded from each of the instrumentation devices depicted in SK-01.
- The strain of any specimen shall be defined as the average reading from the two LVDT devices shown in SK-01.
- Each specimen shall be loaded only until such time as the specimen reaches a strain of 0.71%. Upon reaching this strain the specimen shall be removed from the testing machine.

Preliminary Test

- Specimens A1, A2, and A3 shall be tested when the concrete strength reaches 8,000 psi.
- This test is intended solely to make sure the testing procedure and loading rate are acceptable, and that the data acquisition systems are functioning properly prior to completing the City Test.
- The results of the Preliminary Test shall have no bearing on the decision of SFDBI to allow the use of BauGrid on the 301 Mission Street project.

City Test

- Specimens B1, B2, and B3 shall be tested when the concrete strength reaches 10,000 psi.
- This test shall form the basis for determination of the acceptability of the use of BauGrid on the 301 Mission Street project.

Test Acceptance Criteria

- The City Test shall be deemed successful, and SFDBI shall permit the use of BauGrid for the 301 Mission Street project, if the following criteria are met:
  - Each of the three specimens achieves a strain of at least 0.71%. This corresponds to the beyond-code MCE demand increased to include dispersion.

PROJECT: 301 MISSION	JOB #: 4069	SCALE: 1" = 1'-0"
TITLE: BAUGRID TEST PROCEDURE	DATE: 12/28/2006	DWG. NO.
DESIMONE 160 SANSOME STREET 16TH FLOOR SAN FRANCISCO, CA 94104 - 3722 T. 415.398.5740 F. 415.398.9834	DRAWN: NJR	SK-00
	CHECKED: DDR, RMP	

4 April 2007

Mr. Hanson Tom
City and County of San Francisco
Building Inspection Department
160 Mission Street
San Francisco, CA 94103-2414

Subject: Peer Review Panel Recommendation to Accept Baugrid Reinforcement as Beam Transverse Reinforcement in the 301 Mission Project

Dear Hanson:

We have received the Structural Calculations package dated 22 February 2007, prepared and submitted by DeSimone Consulting Engineers, under the direction of the Derrick Roorda, the Engineer of Record on the 301 Mission Street Project. The package is subtitled *Shear Capacity of Moment Frame Beams Reinforced with BauGrid*, which is the main focus of the package. The package contains a detailed evaluation of the reasons why BauGrids can be accepted as transverse reinforcement in this specific project, including calculations, test data, and opinions from an outside consultant, Murat Saatcioglu, who is an expert in the use of BauGrids.

It is our understanding that the use of BauGrids in the moment frame beams is not being considered as a one-for-one, equivalent replacement of conventional transverse reinforcement. Instead, it is our understanding that the use of BauGrids in the moment frame beams is being proposed on the basis of a performance approach. According to this approach, the use of Baugrids is deemed acceptable if the calculated performance of the buildings is equivalent to or better than the performance anticipated if those buildings were reinforced with conventional transverse reinforcement.

With this understanding, and after review of the information provided in the 22 February 2007 package as well as previous information provided to us about the design of these buildings, it is our opinion that the use of BauGrids in the moment frame beams is acceptable as proposed.

Although we have exercised usual and customary professional care in providing this review, we have not independently verified the accuracy of the calculations provided by DeSimone. Our professional opinions are based on their calculations and further the responsibility of the structural design remains fully with the Engineer of Record.

Respectfully,

Jack P. Moehle

Hardip Pannu

5 January 2007

Mr. Hanson Tom
City and County of San Francisco
Building Inspection Department
160 Mission Street
San Francisco, CA 94103-2414

Subject: Acceptance criteria for tests of Baugrid columns associated with 301 Mission Street project

Dear Hanson:

This letter is to state the position of the undersigned regarding the test specimens, test procedure, and acceptance criteria for Baugrid column tests to be conducted at the Richmond Field Station of the University of California, Berkeley.

The test column geometry is shown in the attached drawing SK-01, dated 10/30/2006. The column test geometry was agreed upon by the undersigned following a review of the geometry of core wall boundary element reinforcement in the 301 Mission Street project, and in consultation with Dr. Murat Saatcioglu, University of Ottawa, who is an expert in the properties and testing of confined concrete columns. We recommend acceptance of this geometry as representative of that in the 301 Mission Street project.

The undersigned also recommend acceptance of the test procedure as described on the attached drawing SK-00, dated 12/28/2006. While we prefer that tests be continued to failure so that we might better understand the limits of behavior of columns made with Baugrids, we accept that this interest in understanding the limits of behavior is outside the scope of this review. Therefore, we are willing to recommend acceptance of the test procedure as described in SK-00.

The undersigned also agree with the acceptance criteria as defined in SK-00, dated 12/28/2006. Our understanding is that the strain limit of 0.71% is based on the strain calculated using the UBC-97 procedure for shear walls, considering orthogonal effects, with displacements amplified by factors **a** and **b**, where factor **a** amplifies the DBE displacement to the expected MCE displacement, and factor **b** amplifies the expected MCE displacement to account for uncertainty in the calculated results. We find this procedure to be acceptable, and therefore recommend that the strain limit 0.71% be accepted. Furthermore, the proposal that all three test specimens reach the strain limit of 0.71% is conservative and we recommend that it also be accepted.

Should the tests pass the acceptance criteria as outlined in SK-00, we recommend that the Department of Building Inspection approve the use of Baugrid reinforcement for columns and walls in the 301 Mission Street project.

Respectfully,

Jack P. Moehle

Hardip Pannu

4 April 2007

Mr. Hanson Tom
City and County of San Francisco
Building Inspection Department
160 Mission Street
San Francisco, CA 94103-2414

Subject: Peer Review Panel Recommendation to Accept Baugrid Reinforcement as Beam Transverse Reinforcement in the 301 Mission Project

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It is our understanding that the use of BauGrids in the moment frame beams is not being considered as a one-for-one, equivalent replacement of conventional transverse reinforcement. Instead, it is our understanding that the use of BauGrids in the moment frame beams is being proposed on the basis of a performance approach. According to this approach, the use of Baugrids is deemed acceptable if the calculated performance of the buildings is equivalent to or better than the performance anticipated if those buildings were reinforced with conventional transverse reinforcement.

With this understanding, and after review of the information provided in the 22 February 2007 package as well as previous information provided to us about the design of these buildings, it is our opinion that the use of BauGrids in the moment frame beams is acceptable as proposed.

Although we have exercised usual and customary professional care in providing this review, we have not independently verified the accuracy of the calculations provided by DeSimone. Our professional opinions are based on their calculations and further the responsibility of the structural design remains fully with the Engineer of Record.

Respectfully,

Jack P. Moehle

Hardip Pannu